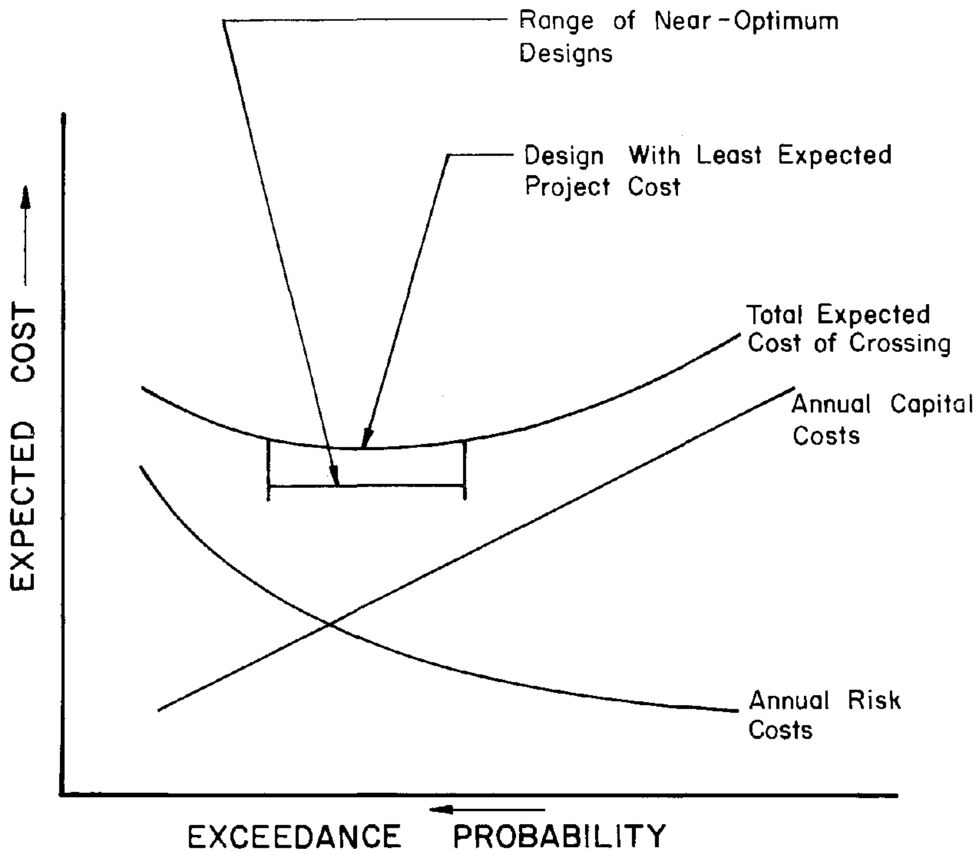
FHWA-TS-80-226

PB81-104259

HYDRAULIC DESIGN OF BRIDGES WITH RISK ANALYSIS



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U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration
Offices of Research and Development
Implementation Division (HDV-21)

March 1980

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BIBLIOGRAPHIC DATA SHEET	1. Report No. FHWA-TS-80-226	2.	3. Recipient's Accession No. PB81 104259
4. Title and Subtitle HYDRAULIC DESIGN OF BRIDGES WITH RISK ANALYSIS			5. Report Date March 1980
			6.
7. Author(s) V. R. Schneider and K. V. Wilson			8. Performing Organization Rept. No.
9. Performing Organization Name and Address U.S. Geological Survey, Water Resources Division Gulf Coast Hydroscience Center National Space Technology Laboratories NSTL Station, MS 39529			10. Project/Task/Work Unit No.
			11. Contract/Grant No. P.O. 5-3-0194
12. Sponsoring Organization Name and Address Federal Highway Administration Office of Development Washington, DC 20590			13. Type of Report & Period Covered Final Report May 1975 - Dec. 1979
			14. P-0258

15. Supplementary Notes

16. Abstracts

This manual presents example studies and reports which implement the philosophy that the total stream crossing including the approach fills in the flood plains and all necessary waterway openings should be designed and constructed for the least total expected cost to the public. The total expected cost includes the capital investment in the highway, expected replacement and repair costs as a result of flood damages, expected user costs from traffic interruption and detour, and expected backwater drainages during the service life of the highway. Techniques for making engineering and economic studies for the least cost designs are presented along with suggestions for managing the time and work required for such studies. A unique design flood is defined for each bridge as the flood whose upstream stage is equal to the lowest elevation of the approach fill or bridge deck. Two example reports were prepared to illustrate the application of the method to a rural site (low backwater damage) and an urban site (high potential traffic interruption and backwater damage).

17. Key Words and Document Analysis. 17a. Descriptors

*Bridge design, Bridges, *Risks, Economic life, Highway effects, *Design flood, Flood drainage, Hydrographs, *Project planning, Design data, Design, Backwater, Flow profiles, Hydraulics, *Hydraulic design, Open channel flow

17b. Identifiers/Open-Ended Terms

Risk analysis, Free board, Example reports, Example studies, Backwater damage, Embankment damage, Traffic interruption, Repair costs

17c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report) UNCLASSIFIED	21. No. of Pages
Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161	20. Security Class (This Page) UNCLASSIFIED	22. Price

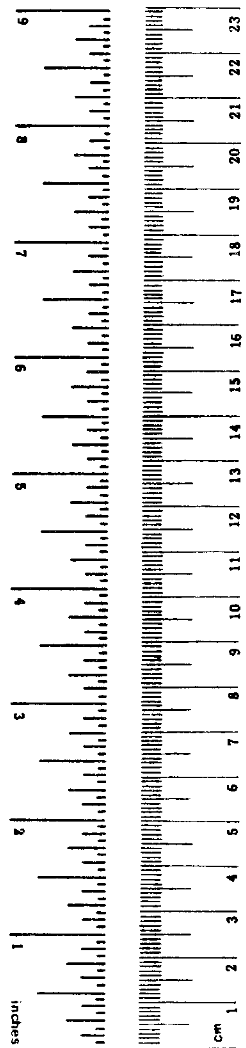
FORM NTIS-35 (REV. 10-73) ENDORSED BY ANSI AND UNESCO. THIS FORM MAY BE REPRODUCED USCOMM-DC 8265-P74

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

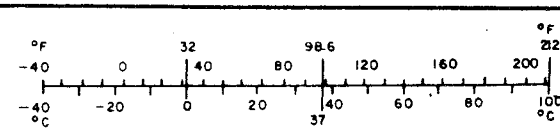
Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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HYDRAULIC DESIGN OF BRIDGES WITH RISK ANALYSIS

By Verne R. Schneider and Kenneth V. Wilson

INTRODUCTION

The hydraulic design of bridges involves an evaluation of the flood hazard to the highway and the effect of the proposed highway on the hazard to lives, property, and stream stability. In specific terms, this evaluation results in the selection of the size and location of bridges, culverts, and other drainage structures and the determination of the embankment fill height. This hydraulic design previously included a flood of a specific frequency where the flood frequency was selected for types of roads depending upon their importance. Structures were designed to safely pass this specific flood, and various safety factors such as embankment freeboard were added to protect the structure and ensure the free passage of traffic under all but the most severe flooding conditions. The effect of the proposed highway on the flood hazard to lives, property, and stream stability were considered using various criteria. Attempts were made to limit inundation of the roadway embankment, to limit embankment erosion and scour, to minimize the backwater caused by the embankment fill, and to minimize the disturbance to the natural stream.

The stream crossing design analysis used in this report is based on the premise that the total stream crossing, including the approach fills in the flood plains and all necessary waterway openings, should be designed and constructed for the least total expected cost to the public. The total expected cost to the public during the service life of the highway includes the capital investment in the highway, expected replacement and repair costs as a result of flood damages, expected user costs from traffic interruptions and detours, and expected backwater damages. Economic and engineering analyses of alternative designs provide information for selecting a design of least total cost to the public or an optimum crossing design.

Briefly, an optimization procedure involves assessing all expected costs during the life of a facility, including capital costs, using a common time frame such as present worth of future expenditure and capital expenditures or annual cost of capital investment and expected future annual expenditures, and choosing the alternative which costs the least. Thus, the least total cost facility is the optimum solution and makes the most effective use of public monies. To illustrate, in figure 1,

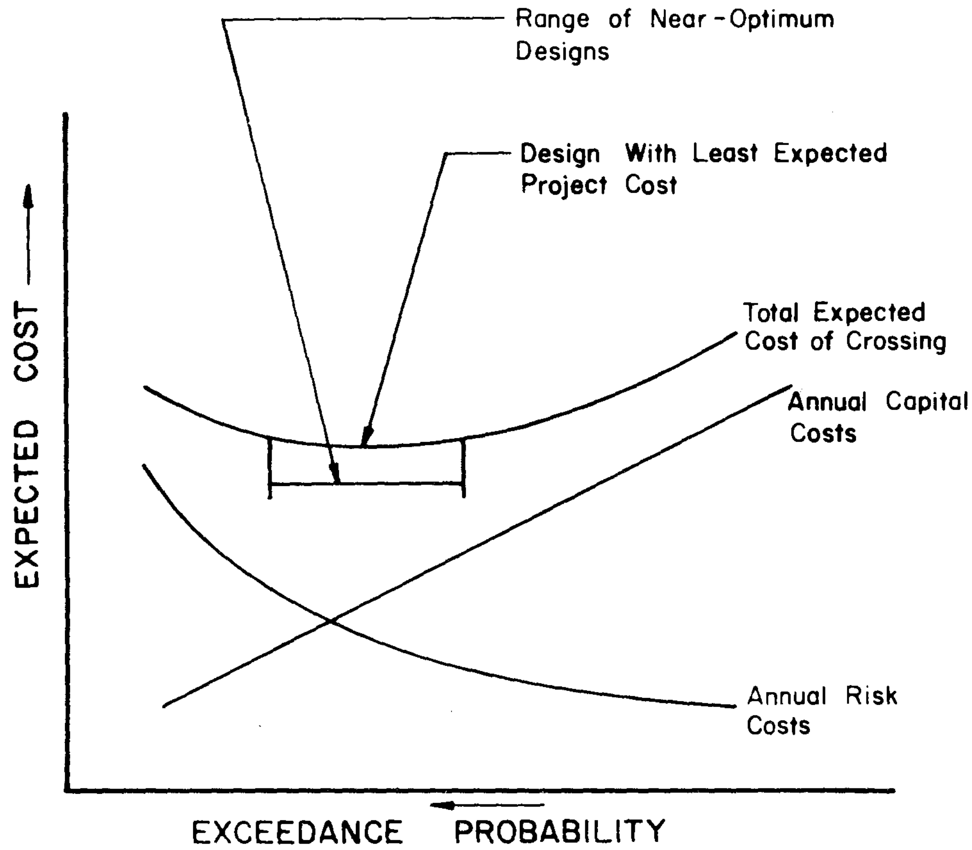

FIGURE 1. VARIATION OF EXPECTED COST WITH EXCEEDANCE PROBABILITY.

assume that the required capital investment for a stream crossing increases as the exceedence probability of the design flood is decreased. Assume also that annual maintenance costs, user costs, damages from backwater, and repair and replacement costs decrease with designs for smaller exceedence probabilities. Summation of the annual capital costs and annual maintenance and operations costs yields the total annual costs for any given exceedence probability. A graphical presentation of these costs versus exceedence probability provides a basis for making a rational design selection.

Techniques for making engineering and economic studies for the least cost or optimum designs are presented in this report as well as suggestions for minimizing the time and work required for such studies. Procedures for developing engineering information, such as water-surface elevation-discharge relations, are not included.

This report begins with a chapter on the mechanics of the study. A design philosophy is discussed which allows a designer to determine the optimum design. The concept of risk analysis is introduced as a technique to evaluate the relationship between construction costs, probable future damage to highways, future damage to other property associated with the highway encroachment on the flood plain, and the costs for traffic using alternative routes and detours. Budgetary constraints, the need for emergency supply and evacuation routes, and the need for emergency vehicle routes either may be included in the risk analyses to the extent that their value can be stated in dollars or may be considered in conjunction with the risk analysis. The analysis procedures are then summarized as a 16-step process culminating in a report. A suggested report outline is included.

Next, descriptions of the data collection and analysis procedures are included in the chapter on the report of the evaluation. This chapter generally follows the suggested report outline and the outline of the two example studies which are included as Appendices A and B. The two example studies include a rural site where the potential for backwater damage is low and traffic volumes are moderate, and an urban site where the potential for backwater damage is high and the traffic volume is heavy.

This report and the two example studies do not cover every circumstance and condition that occur in the design of encroachment. The study framework is sufficiently general so that situations not covered can be easily incorporated.

MECHANICS OF STUDY

The purpose of this chapter is to discuss the design philosophy used in this report, to define risk analysis, to clarify some design concepts as they apply to risk analysis, to outline the procedure used, and to suggest one possible report outline.

Design Philosophy

Hydraulic design with risk analysis incorporates economic risks directly into the design process. Economic risks are expected losses that can be divided into three categories: (1) direct damage to the roadway and bridge, (2) traffic related losses, and (3) losses due to additional flood damage in the upstream flood plain. Traditionally, the hydraulic design of bridges has been based on hydrologic and hydraulic considerations with economics incorporated in a very indirect manner.

The design philosophy recognizes that no single discharge can characterize the random stresses that nature will impose on a bridge site; thus, it is necessary to evaluate expected losses from the full flood-frequency curve by applying the appropriate exceedance probability for each discharge on the curve. The philosophy presumes that all the losses can be quantified and that an optimal design can be found by minimizing the sum of expected losses and capital costs expressed on an equivalent economic basis.

The design techniques associated with this philosophy are conceptually simple. Techniques involve: (1) trying several bridge designs, (2) selecting several discrete discharges to adequately represent the flood frequency curve, (3) repetitively making hydraulic and economic computations with each discharge for a given alternate design, and (4) ultimately, selecting the alternate that has the least total cost to society considering expected losses as well as capital costs.

Several notions about design require elaboration to further illustrate the implications of this design philosophy. These notions include the so called design discharge, the base flood, freeboard, and economic analysis which are discussed in detail below.

The results of the risk analysis are one component of the decision making process. The decision making process will also include consideration of budgetary constraints, the need for emergency supply and evacuation routes, and the need for emergency vehicle access. The provisions of the Flood Insurance Act may also apply to the design process. The risk analysis, however, provides a means of assessing the cost of such constraints by comparing the cost of the encroachment with the constraints to the cost of the alternate that has the least total cost to society.

Design Discharge

Understanding the new role of design discharge is one of the most difficult tasks in making the transition from traditional design to risk-analysis design. Traditionally, designers start with a semi-arbitrary design discharge, select a bridge opening that will pass this discharge, and add freeboard to computed water-surface elevations to establish embankment and low bridge steel elevations. In risk analysis the design discharge for any combination of embankment and bridge opening size is the discharge whose elevation upstream of the encroachment is equal to the minimum elevation of the encroachment. (In other words, it is on the verge of overtopping the highway.) In risk analysis, there is no one discharge that dictates the design, and the design discharge is one of the results of the analysis. Since the risk analysis described in this report is oriented towards traffic requirements, design discharge that results from the risk analysis is an indicator for impending traffic interruptions.

Designers may, in fact, choose to make several preliminary traditional sets of computations based on arbitrary discharges to determine some of the trial bridge designs to be used in the risk analysis. Such computations are, however, side issues to the risk analysis because trial bridge designs could be selected by other methods. The final design discharge that results from the risk analysis may or may not correspond to any of the discharges used in the preliminary computations.

Risk analysis can be applied to many aspects of design. This report is oriented to determine embankment elevations and bridge lengths, but the concepts are applicable to other components of design. Other components of design that could be determined by risk analysis include scour depths used to establish foundation elevations, clearance requirements for low steel elevations on bridges, and protection measures. Each of these components may be associated with design discharges that are quite different from one another. For example, Laursen (1970) made a sensitivity risk analysis to illustrate that scour estimates should be based on the maximum flood that might occur because the results of losing a typical bridge are so catastrophic. In other words, the design discharge associated with scour computations may have a very low exceedance probability (say .0001) in a given year; whereas, the design discharge associated with establishing the embankment elevation and bridge length may have a relatively higher exceedance probability (say .04) in a given year.

The Base Flood

The base flood (1-percent-chance flood) is referred to in Executive Order 11988--Floodplain Management. Executive Order 11988 is intended to avoid to the extent possible adverse impacts associated with occupancy and modification of flood plains. Flood plains, as a mimimum, include areas subject to inundation by the base flood.

The base flood should be within the range of discharges selected to make the risk analysis, but it will not necessarily be the final design discharge in any case. The base flood will not necessarily be the largest flood considered in the risk analysis.

Freeboard

Freeboard is a common practice of adding elevation to the embankment. Embankment freeboard is perceived as a safety factor to prevent occasional saturation of the base course and to reduce expected traffic interruption. While embankment freeboard does reduce expected traffic related losses and perhaps expected highway damages, it increases expected losses due to additional flooding of the upstream flood plain.

While freeboard is commonly added at the end in traditional design, it is included in the trial designs for a risk analysis. In risk analysis, expected traffic losses play a predominate part in the optimization procedure; therefore, applying freeboard to the embankment would simply be a means of introducing another trial design for consideration. Occasional saturation of the base course, furthermore, is just another component of expected highway damages. Although the occasional saturation component was ignored for the examples in this report, it could certainly be included at a designers discretion.

In summary, then, embankment freeboard has no meaning in risk analysis since embankment elevation is one of the variables in the analysis. Risk analysis provides the basis for selecting the most economical elevation, and it provides the basis for evaluating the cost of selecting an elevation other than the optimum. A designer can certainly select an embankment elevation that is greater than the most economical one. The biggest deviation from tradition is the design discharge which is unique to the elevation that is finally selected.

Economic Analysis

Every assessable cost or damage is included in the economic analysis. Individual costs can be examined in the decision making process to determine whether they are reasonable or whether minor design changes might be made to reduce risk further. An assumption that influences the economic analysis is that damages will be repaired so that they have the same opportunity to recur year after year. This assumption means that the probability of a given risk is the probability, p, that a given flood will occur in a year. Actually, there is chance that a given flood may occur several times during the life of a structure. The probability that a given flood occurs at least once during the life of a structure is $[1 - (1 - p)^n]$ where n is the service life of the structure.

All computations were made in terms of constant dollars. Hence, only the real cost of capital should be represented in the interest rate used. This interest rate includes no allowance for expected inflation. Such a procedure allows the analyses to be made using prices that prevail when the economic study is made. A more complete discussion of the assumptions and limitations is included in Howe (1971).

In other words, while the going interest rate might be around 10 or 11 percent, the interest rate that should be used in these analyses might be around 5 percent. Interest rates normally do include a perceived inflation factor. By using this rate instead of the prevailing interest rates, the economist does not have to apply inflation factors to replacement costs that may not be incurred until some time in the future. All costs can then be estimated at today's prices.

Determining just what the interest rate should be is beyond the scope of this report. For illustrative purposes, an interest rate of 5 percent has been used throughout this report.

Risk Analysis

Definition

There are many risks or uncertainties attached to the development, design, construction, and use of a project. In this report, an attempt is made to quantify the risk associated with the damage to the embankment when overtopped, damage caused by backwater, cost of traffic interruption when the embankment is overtopped, and other quantifiable costs which may be appropriate to the site. Even so, there is an uncertainty associated with each item; for example, the traffic volumes may be higher or lower than estimated.

To obtain the total expected damage, the relation between damage and all possible floods which would cause damage is computed. A typical computation is shown in table 1. The trapezoidal rule is used to compute the increment of expected damage between each flood chosen for computing damages. Risk is computed as the product of the exceedance probability interval (column 13) and the average damage for interval (column 11). The damages for floods larger than the largest flood (labeled ultimate in table 1, the 0.2-percent-chance flood) are assumed to be the same as those for the largest flood. The expected damage of the largest flood is the damage of the largest flood multiplied by the exceedance probability of the largest flood.

Table 1.--Summary of risk analysis

Site: Leaf River near Collins, Miss.

Trial Design Discharge: 49,000 ft^3/s

Embankment Height: 231 feet

Bridge Deck Length: 480 feet

Estimated Capital Cost: $1,342,000

Estimated Service Life: 25 years

Interest Rate: 5 percent

Annual Capital Cost: $95,215

Flood ft^3/s	Flood probability	Maximum depth over roadway, feet	Duration of flow over the road, hours	Time road is closed for repairs, hours	Cost in dollars: Traffic interruption	Cost in dollars: Backwater damage	Cost in dollars: Overtopping damage to embankment	Cost in dollars: Other damage	Total damage, dollars	Average damage for interval, dollars	Probability interval, ΔP	Risk, dollars
37,000	0.048					0			0	100	0.008	1
39,000	.040					200			200	750	.007	5
41,000	.033					1,300			1,300	1,850	.004	7
43,000	.029					2,400			2,400	3,200	.004	13
44,400	.025					4,000			4,000	4,100	.005	20
47,200	.020					4,200			4,200	9,574	.006	57
51,500	.014	0.4	14		6,649	6,300	2,000		14,949	27,080	.002	54
54,200	.012	.9	22		10,449	9,762	19,000		39,211	46,426	.002	93
56,100	.010	1.2	27		12,824	9,318	31,500		53,642	75,300	.0033	248
62,000	.0067	2.2	40		18,998	7,961	70,000		96,959	115,878	.017	194
66,000	.005	2.8	48		22,798	0	112,000		134,798	193,845	.003	582
80,000	.002	4.6	68	96	77,893	0	175,000		252,893	252,893	.002	506
Ultimate			68	96	77,893	0	175,000		252,893			

Total Annual Risk	$ 1,780
Annual Capital Cost	$95,215
Total Expected Project Cost	$96,995

Usefulness of Risk Analysis

Risk analysis provides a means to quantify in dollars the flood hazard to the highway and the effect of the proposed highway to lives, property, and stream stability. Since both risk and construction costs depend upon the bridge and embankment, the sum of risk and the annual construction cost (total construction cost multiplied by the capital recovery factor) measures the true cost of the flood hazard to the highway and the effect of the proposed highway to lives, property, and stream stability. (Other costs such as land acquisition and engineering design have not been included in the analysis because they are assumed to be constant one-time costs which are not usually subject to the flood hazard.) The most economical design would theoretically be the bridge and embankment with the lowest cost.

A risk analysis is a useful design analysis tool in that it focuses the designer's attention on the risk to the structure caused by flowing water and the risk to items such as traffic, property, and the river channel caused by the structure in relation to the actual construction cost. The designer may find that the least costly structure is one that just spans the river at valley level and may be occasionally overtopped by a low frequency flood.

Risk may be reduced by spending additional money on the highway structure. For example, a longer bridge which spans the valley and contracts the flood less will reduce the amount of backwater caused by the bridge. This in turn will reduce the flooding of property which may be located on the flood plain and which would not be flooded had the bridge not been there. Risk analysis allows the designer to determine the significance of such damages. When these damages are significant, the designer may choose to adjust the fill height, add spans to the bridge, or perhaps use channel protection. The annual cost of such additions should be computed and compared to the risk that they are intended to prevent. The annual cost of additions in general, should not exceed the expected cost due to risk. For example, adding spur dikes to a bridge may cost $5,000 or $354 per year (5 percent interest, 25 years life). Assume that an abutment washout would be anticipated with a 5 percent (20 yr.) flood. The spur dike itself might fail in a higher flood (say a 0.5 percent flood) but it effectively reduces the probability of a washout from 5 percent to 0.5 percent ($\Delta p = .045$). Furthermore the spur dikes will probably require repairs (say 0 loss for a 5-year flood, 10 percent loss for a 10-year flood, and 100 percent loss for a 200-year flood) which are equivalent to $312 year. ($\$5000\ ((0 + .1)/2\ (.2 - .1) + (.1 + 1.)/2\ (.1 - .005) + 1.0(.005)) = \$312$). The total annual cost of spur dikes is $354 + $312 = $666 which is justified if the expected damages - cost of repairs, traffic losses, and potential loss of lives - from an a abutment washout (without spur dikes) exceeds $14,800 since $14,800(0.045) = $666.

Procedure

The hydraulic design of bridges with risk analysis will consist of several steps:

1. Define and understand the project.
2. Select analytical procedures for computing water-surface elevations, flow distribution, flood frequency, and discharge and stage hydrographs. Collect the field data necessary to use these procedures.
3. Compute the natural water-surface profile, lateral flow distribution, and peak-discharge-frequency relation.
4. Define the combination of bridge deck lengths and embankment elevations which could conceivably be built.
5. Do a hydraulic analysis for each structural combination to be considered. Compute the expected water-surface profile, the design discharge, the velocity of flow through the bridge, and the distribution of flow over the road.
6. Determine the cost of rerouting traffic in case of an embankment overtopping or washout. Compute the overtopping time and depth of flow and estimate the cost of traffic interruption.
7. Evaluate the flood damage due to backwater by computing the difference between the damage with the bridge in place and the natural condition.
8. Evaluate the potential for embankment damage and estimate the cost to repair damage that might occur.
9. Evaluate the capital costs that will vary in the analysis. Include costs for foundations in alluvial beds and protection measures which are particularly dependent upon embankment elevation and bridge length. Exclude costs for right-of-way and design which will not vary appreciably in the analysis. Devices added to prevent risks are added to construction costs. Decide on a service life for the structure and an interest rate. Compute the annual capital costs.
10. Evaluate the cost of any other potential damages which may be unique to the site such as structural damage, embankment costs, debris, ice, and scour.
11. Compute the risk.
12. Add the annual construction cost and risk to obtain the total expected project cost.
13. Plot two families of curves as illustrated conceptually in figure 2. In one case, plot total expected cost versus embankment elevation. In the other case, plot design discharge and return interval versus embankment elevation.

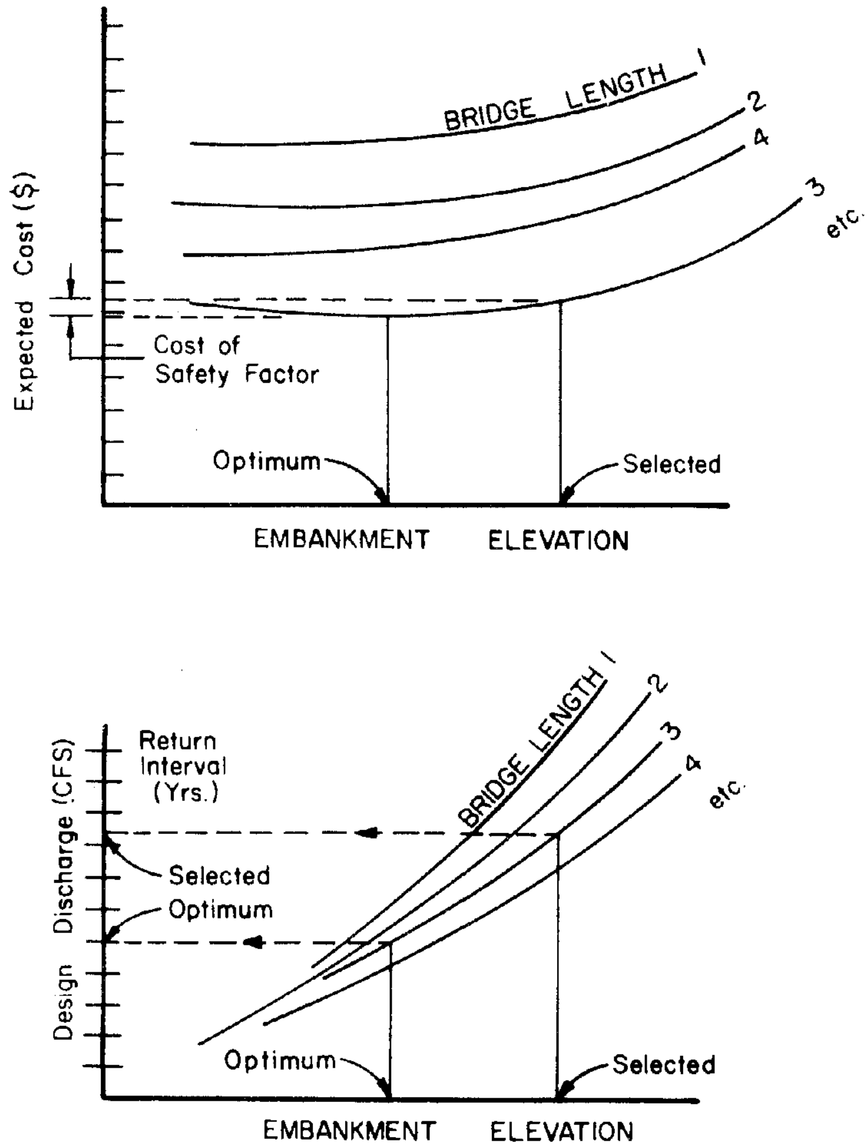

FIGURE 2. SUMMARY OF THE RESULTS OF THE RISK ANALYSIS AND THE SELECTION OF THE DESIGN DISCHARGE, BRIDGE LENGTH, AND EMBANKMENT ELEVATION.

14. Select a bridge length and embankment elevation combination. Determine the corresponding design discharge and return interval. The optimum combination defines the lowest point on the lowest curve of total expected cost versus embankment elevation. The curves are often relatively flat and the selected combination may provide considerable safety factor against traffic interruption without much additional cost.

15. Evaluate the cost of the safety factor, if any.

16. Document the results in a report.

The entire analysis could be computerized. Tseng and others (1975) have published a program for the purpose. However, once a computer program is constructed, it locks in some analytical techniques which may vary among analysts. This report approaches the analysis in blocks. The hydrology, hydraulics, construction costs, traffic costs, and flood damage could and, in fact, did come from separate sources. There are no known verified techniques for estimating damage to the embankment due to flow over the road. Therefore, one was devised using a couple of experience points and engineering judgment for the purposes of this report.

While much of the report can be easily developed by hand with the assistance of a calculator, other parts may be expedited with a computer. These include the computation of water-surface profiles when step backwater or more sophisticated techniques are used. The bookkeeping involved in conducting a depth-damage analysis can be simplified with the use of a computer program. In this report, a computer program was available for the urban site which computed the damage and risk due to flood-plain inundation. For traffic and embankment erosion, the damages for each component flood were computed. The techniques used and the computer model selected are not important so long as they are sufficiently accurate.

THE REPORT

Report Outline

A suggested outline for a report to document the results is shown in table 2. The two sample reports are included in appendices A and B and are written in a narrative form. In this chapter of this manual, each topic outlined in table 2 is annotated describing techniques and sources of information. In this way, the reader may refer to corresponding sections in the two example problems. Although the sample reports are written in narrative form, it seems possible that tables, graphs, computer printouts, check sheets, or drawings might be substituted for parts of the analysis in order to minimize the time involved in preparing a report.

Table 2.--Typical Report Outline

INTRODUCTION		
	Acknowledgment	
	Purpose	
	Project Description	
DATA COLLECTION		
	Flood Plain and Channel Geometry	
	Land Use	
	Hydraulic and Hydrologic Data	
	Geologic Data	
		Surface Geology
		Channel Morphology
		Soils Information
		Scour History
	Cost Data	
		Capital Cost
		Backwater Damage
		Traffic Interruption
		Embankment Repairs
DATA ANALYSES		
	Magnitude and Frequency of Floods	
	Hydrographs	
	Natural Water-Surface Profiles	
	Flow Distribution	
	Special Considerations	
	Alternative Bridges Considered in the Risk Analysis	
	Water-Surface Profiles	
RISK ANALYSIS		
SUMMARY AND CONCLUSIONS		

The report can be organized into five major sections as shown in table 2. These include an Introduction, Data Collection, Data Analysis, Risk Analysis, and Summary and Conclusions sections. Each section can be subdivided into subsections depending on the site and the type of information being presented. This chapter describes each of the sections and includes suggestions on the type of information to be included.

Introduction

The principal features of the project are described in the Introduction. The Introduction is divided into three subsections in table 2, allowing for an Acknowledgment, Purpose of the Study, and a Project Description. More subsections may be added as needed.

Acknowledgment

In the Acknowledgment subsection, the agency conducting the study or for whom the study is being conducted, the contract number or the authority under which the study is being conducted, can be identified. Agencies and persons who contributed data used in the report should be acknowledged. The results of previous studies can be cited.

Purpose

The purpose of the study contains a statement about the information and analyses being furnished by the report. These might include references to the work, statement of a contract, or other agreement.

Project Description

Sufficient information should be included in a project description so that the reader might be informed of the location and scope of the project, where this bridge is to be built, how long and wide, how many lanes, and the approximate valley width. Location maps, plans and (or) aerial photographs may aid in describing the project.

Data Collection

Data are collected for use in specific computational procedures. Since data are expensive to collect, the investigator will probably want only those data needed to complete the study. These data and how they are collected are described in this section. They should, however, satisfy the techniques used in the data analyses.

Flood Plain and Channel Geometry

Cross sections of the valley are surveyed for use in computing the water-surface profiles where a one-dimensional step-backwater technique is used. When two-dimensional flow models are used, more detailed data may be needed. The computational methods chosen govern the amount of data necessary.

Land Use

Land use is an important variable in computing backwater damages. A survey will quantify the types and values of the various structures (houses, warehouses, factories, and so forth) and other uses (agriculture, recreation, and so forth) of the land which may be inundated by flood waters.

Hydraulic and Hydrologic Data

Summarize and discuss the source and availability of hydraulic information pertinent to the site. These data may include stage-discharge relations, measured water-surface elevations from previous floods, velocity distributions measured at an old bridge and discharge measurements. Earlier site reports may be available which summarize this information. Manning's n value should be selected for each section or subsection to be used in the hydraulic analysis. Measured water-surface elevations should be used to calibrate and verify the flow model selected to compute the water-surface profiles.

Hydrologic data may be developed from gaged records at the site or from nearby sites. A discharge frequency curve will be constructed and a discharge hydrograph constructed for each discharge used in the analysis. The discharge frequency relation is used in computing the risk while the hydrograph is used to obtain the length of time that an embankment might be overtopped (and traffic interrupted) during a flood. The overtopping time may also be useful in evaluating embankment erosion potential. The degree of local scour may depend on the duration of the hydrograph.

Geologic Data

Several types of geologic data may be useful in the bridge site investigation. These include surface geology, channel morphology, soils information, and scour history.

Surface Geology.--Surface geology supplies information on the regional geological conditions. The surface formation can be determined and characteristics of the formation related to bridge designs could be specified.

Channel Morphology.--Channel morphology is a complex problem (Richardson and others, 1975) which can affect the stability of bridges in an uncertain manner. In this section, describe the development of meanders, bank erosion, channel braiding, head cutting, general scour and deposition that is not related to the bridge, channel regulation, and other factors. Channel straightening to improve the flow alinement through the bridge can result in bank erosion in some soils and bank materials. Proper bank protection may be needed. Evolving channel conditions should be considered if there is a chance that the channel will change significantly over the design life of the structure.

Soils Information.--Soils information should be collected by drilling a system of holes to elevations well below probable bridge foundation. Unusual foundation conditions can increase the initial cost of the bridge. Potential severe scour conditions can cause the need for special foundation designs to enhance the stability of the bridge. The scour depth, and hence failure due to scour, is difficult to predict even under conditions similar to those studied in laboratories. The objective in this section is to identify conditions which should be considered and to state the assumptions made.

Scour History.--Some bridges are replacements for existing obsolete or failed structures. Others cross streams already crossed at other locations. Hence, for some sites or nearby sites there is a scour history. Although the data may not be quantitative or complete, this experience should be summarized to provide a basis for design. Avoiding certain conditions may be the best countermeasure for scour. Estimate the parameters being used to determine where scour will occur. For example, determine the maximum velocity and the velocity distribution. Even though scour might never have caused a problem at an existing site, the current centerline profile could be compared to the profile at the time of the bridge construction. An obsolete bridge being replaced could be on the verge of failure caused by scour.

Cost Data

Cost is the dependent variable in the analyses. One needs to know the cost of construction, backwater damage, traffic interruption, and embankment repairs if they were needed. Maintenance costs would be needed if the cost of maintaining the various assumed bridge/embankment combinations are significantly different.

Capital Cost.--The biggest single cost item is normally the capital cost of the project. Various procedures are used by highway agencies to estimate these costs. For the purpose of this report, these costs were obtained from the Highway Department. Capital costs increase with bridge length and embankment height as shown in figure 3.

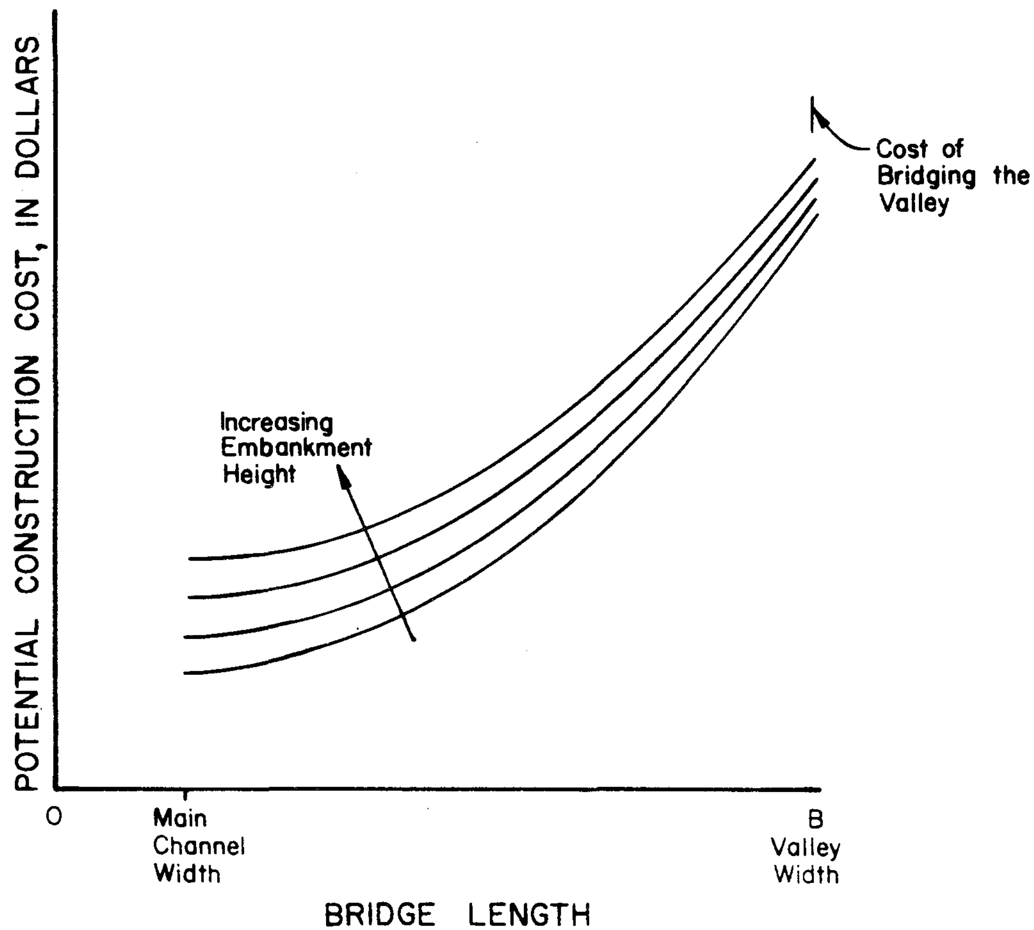

FIGURE 3. VARIATION OF CONSTRUCTION COSTS WITH BRIDGE LENGTH AND EMBANKMENT HEIGHT.

The capital costs, however, are initial costs that must be amoritized over the service life of the structure. The annual amoritization series is determined by multiplying the capital costs by the capital recovery factor, CRF. The CRF is defined as "an annuity whose present value is one;" it can be computed from

$$CRF = \frac{i(1 + i)^n}{(1 + i)^n - 1} \tag{1}$$

where

i = interest rate

n = service life of the structure

The capital recovery factor is tabulated in most interest tables. (See, for example, C. D. Hodgman, 1957, pp. 427-434.)

Backwater Damage.--Backwater is the increase in upstream water-surface elevations required to force the discharge through the bridge and, at times, over the embankment.

Backwater usually causes more land to flood compared to the natural condition. On land flooded naturally, backwater causes flooding to be deeper. The damages associated with increased flooding may be computed from data obtained during a land use survey and from stage damage relations. The upstream valley which may be affected by backwater should be surveyed to determine the land use. At many sites, there are a limited number of land uses that must be identified. The Corps of Engineers (1977), Sumrall (1970, 1972), G. T. McDonald (1977, written commun.), F. V. Reilly (1978, written commun.), and others have procedures to develop stage damage relations for various land uses.

A survey adequate to satisfy the requirements of a risk analysis is as follows:

1. Estimate the reach that would be affected by backwater.
2. Identify features, such as residences, schools, warehouses, fields, and woods that can be assigned a value as a unit.
3. Determine the elevation at which damage from flooding first occurs such as the floor elevation of a house or the typical elevation of a field. Larger fields may have to be split into several subareas when the elevation changes significantly.

Table 3 could be a useful guide in tabulating the data. The data can be either used for hand computation or by computer.

Table 3.--Summary of typical data collected during a land-use survey

Item number	Description	Current use	Initial damage elevation (ft)	Location on flood plain (station) (ft)	Value	Area	Remarks
1	Jones' shed	Store farm equipment	272.6	5210	\$500	400 ft^2	
2	Field	Agriculture--small grains	268.9	5620		2 acres	Rotated between soy beans and corn
3	Field	Pasture	267.1	6320		5 acres	
4	Smith's house	Residence	274.8	6500	\$25,000	1500 ft^2	

The difference between the flood damage with the bridge in place and the flood damage under natural conditions is the damage caused by backwater flooding. The depth-damage relation for types of land use is used to estimate the damage. For example, water 1 foot deep in a house valued at $20,000 might cause $4,550 damage while 2 ft could cause $6,300 damage. The difference between the water-surface elevation and the initial damage elevation is the depth of water at that point. The depth-damage relation is used to obtain the average value of the damage. Depth-damage relations have been developed by FEMA (Flood Emergency Management Agency) (F. V. Reilly, 1978, written commun.), the Corps of Engineers (see Sumrall, 1970), and others as a result of damage surveys and preplanning activities. The classification of structures and land uses are being refined by FEMA as claims experience develops. As these damage values are based on actual experience, it is recommended that FEMA data be used to compute flood damage.

One computational procedure was developed by Sumrall (1970, 1972). In this approach, the value of the structure and the floor elevation are the input data. Damage is a percentage of the structural value and varies with the depth of flooding above the floor. Building contents are specified as a percentage of the structural value, but for estimating purposes are taken as 50 percent of the structural value. Water-surface elevation frequency information is obtained from the hydraulic analysis.

The computations of depth damage are conceptually simple and can be hand calculated. However, they are also laborious and the use of a computer program is recommended.

Backwater damage (fig. 4) will increase with embankment elevation for a constant bridge length because more water is forced to flow under the bridge and less is allowed to flow over the embankment. However, as bridge length increases more water flows under the bridge, relieving the backwater. Therefore, backwater damages decrease as bridge length increases and increase as embankment elevation increases. For a given discharge, when all flow goes through the bridge, no additional backwater increase will occur.

Traffic Interruption.--When the road is out of service either by overtopping or structural failure, the difference between the cost of driving a vehicle on the primary detour and the usual route is considered to be part of the risk involved in the design. These running costs are available from the state transportation agency. Considerable knowledge of the road is required to compute running costs. Necessary data includes the traffic volumes, value of time lost, vehicle running costs, basic geometrics of the section of road, unit accident costs, length of the section, transition costs owing to changes in speed along the route, and delay costs. These data are used in doing cost benefit analyses for road improvements.

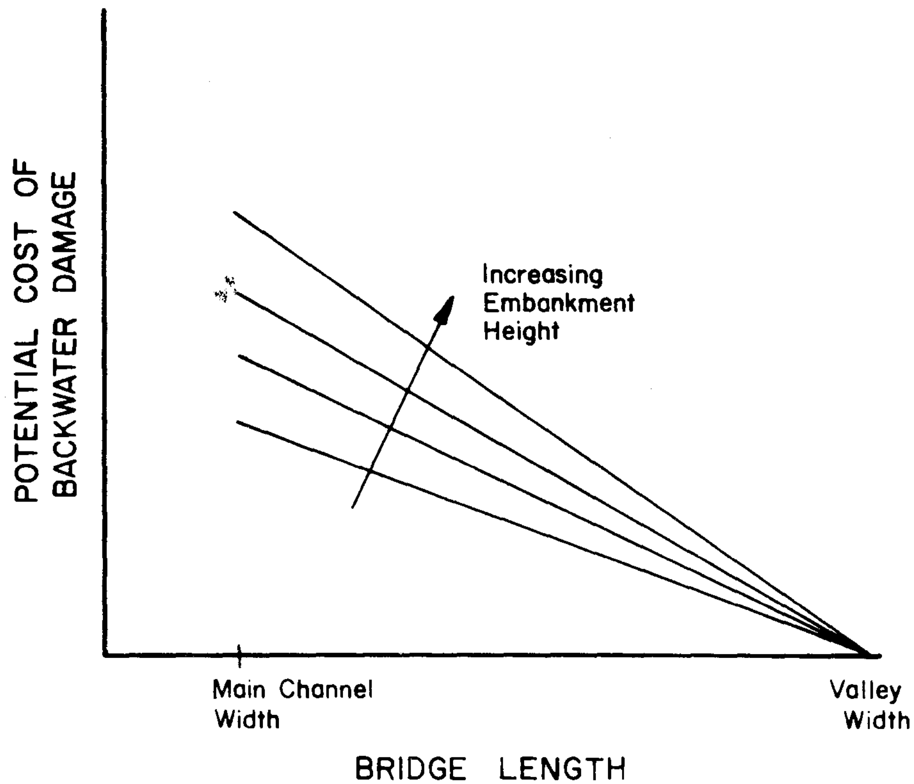

FIGURE 4. VARIATION OF BACKWATER DAMAGE WITH BRIDGE LENGTH AND EMBANKMENT HEIGHT.

The difficulty with the procedures such as those published by Anderson and others (1975) and Tseng (1975) is that the unit costs used in developing the regression equations and monographs increase rapidly. In many cases, the risk component due to traffic will be small. Where severe grades, long detours, or extreme speed differences between the main road and detour exist, a detailed evaluation of running costs may be in order.

A reasonable approximation of the cost of the detour can be determined by using the measured traffic volume multiplied by a vehicle cost per mile and the added mileage. FHWA annually publishes figures on the vehicle cost per mile for the various classes of vehicles. From these figures, the cost per mile can be determined for the various traffic mixes.

Traffic normally increases from an initial low volume to a projected high volume. For purposes of the economic analysis, it is reasonable to assume that the traffic volume will be a gradually varying series. A gradually varying series can be converted to an equivalent uniform annual series by procedures described by Grant and Ireson (1960). The equivalent uniform annual series for the average daily traffic, ADTE, that represents a growing traffic series is:

$$ADTE = ADT1 + G(gf) \tag{2}$$

where

ADT1 = initial ADT at the end of the first year

G = growth rate of the traffic volume (ADTN - ADT1)/n

ADTN = projected ADT at the end of n years

gf = factor to convert a gradually varying series to an equivalent annual series

$$gf = \frac{1}{i} - \frac{n}{(1 + i)^n - 1}$$

i = interest rate

Since traffic may be interrupted due to flow over the road, user cost decreases as embankment height increases as shown in figure 5. In general, as bridge length increases, more water will go through the bridge and less over the road. Therefore, cost of traffic interruption will decrease as bridge length increases. For a given discharge, when all the flow goes through the bridge, no traffic interruption will occur.

<u>Embankment Repairs</u>.--When road fills are overtopped, some erosion occurs. The amount of erosion is not predictable by available methods. However, there is the question of how much it will cost to replace sections of the embankment if it does wash out. The availability of fill material, haul distance, the cost of equipment, and even the cost of placing temporary structures should be considered. In these studies embankment erosion was assumed to vary with duration and depth of embankment overflow.

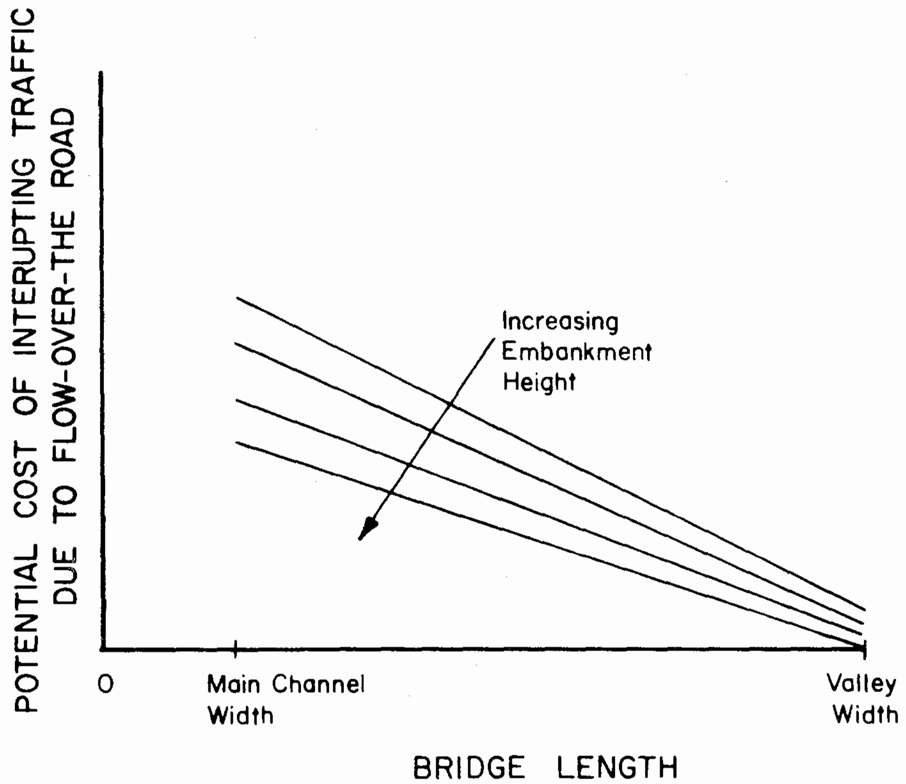

FIGURE 5. VARIATION OF TRAFFIC INTERRUPTION COSTS WITH BRIDGE LENGTH AND EMBANKMENT HEIGHT.

The chance of embankment overflow is smaller for the higher embankments. Therefore, the cost to repair damages should decrease (fig. 6) with increasing bridge lengths and increasing embankment elevations.

Data Analyses

Magnitude and Frequency of Floods

Flood frequency is computed using the best available procedure using national, regional, or state reports. At this time, an extensive set of reports has been written by the U.S. Geological Survey covering flood-frequency analysis in each of the various states. These include both large and small watersheds. The United States Water Resources Council (1977) has published a procedure for developing flood frequency at gaged watersheds and is currently working on recommendations for ungaged sites. Various state agencies have developed methods for use within the state. The sources of this information are well known to hydrologists and hydraulic engineers within the state and is generally available from the U.S. Geological Survey's district office.

Once a flood-frequency relation has been computed, a series of floods can be selected for use in the risk analysis (column 1 of table 1). In the risk analysis, a numerical integration is performed involving floods of various frequencies. The more floods that are used the more accurate the integration. However, the number of computations increase. The floods are selected from the flood-frequency relation for various exceedance probabilities.

1. The first flood is one at which, if exceeded, some type of damage will occur such as backwater, traffic delay, or erosion damage. It will probably be at least the bankfull stage.

2. The base flood, the 1-percent-chance flood, is used because the entire stream crossing--the bridge, embankment and roadway--is required to pass the 1-percent-chance flood.

3. Select at least two intermediate floods between the first flood and the 1-percent-chance flood.

4. Select several large floods which would be expected to cause damage in high risk areas that include critical facilities such as hospitals, rest homes, and so forth. Such floods could cause major traffic delays and incur large costs in repairing the highway crossing. The 0.2- and 0.5-percent floods are usually such floods.

Hydrographs

In addition to the selection of a sequence of peak discharges, a flood hydrograph with that peak is needed to compute the duration of flow over the road. In some instances the local scour potential is increased for the larger hydrographs, that is, the longer the water is up the more extensive and severe the local scour might be.

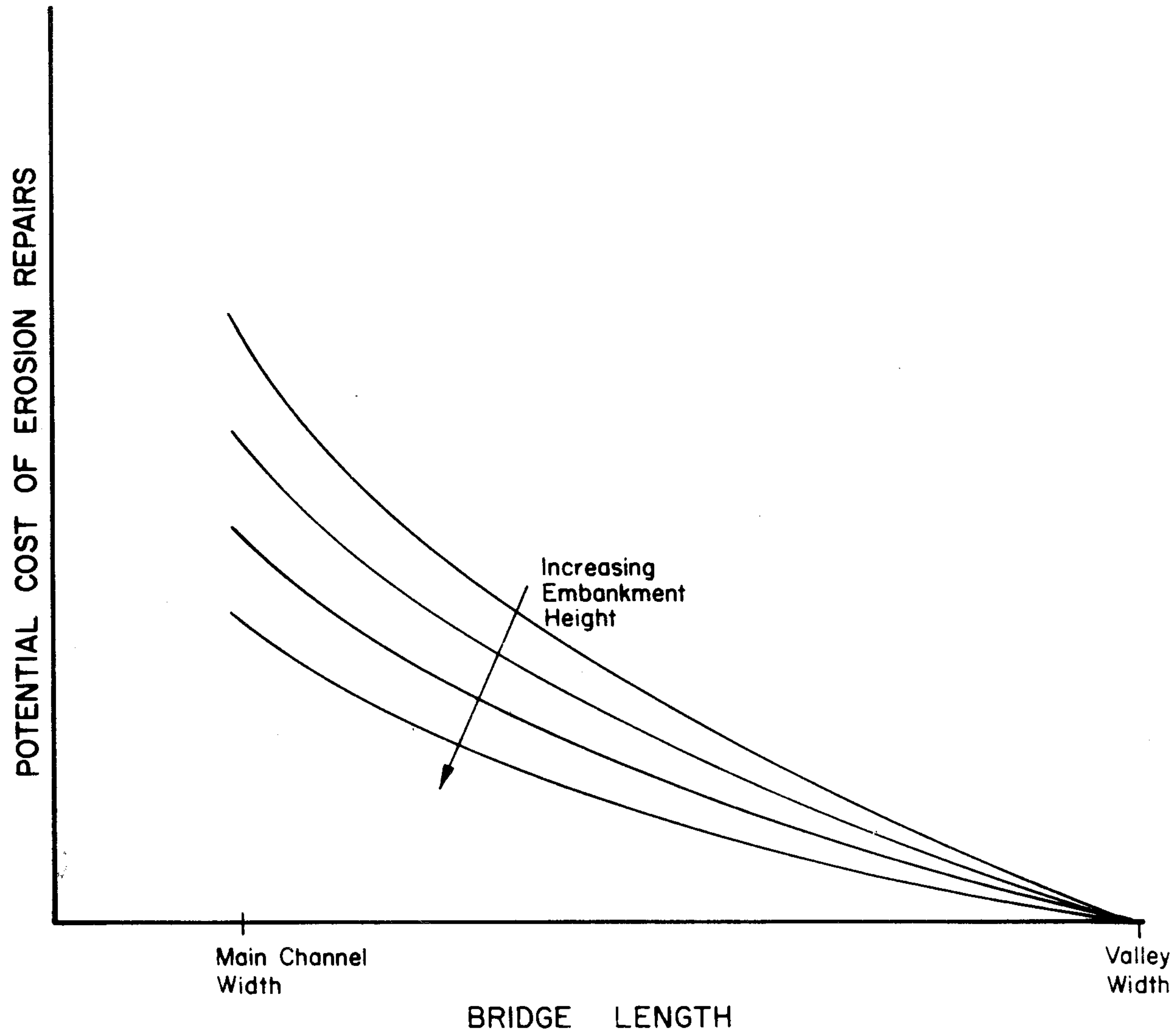

FIGURE 6. VARIATION OF EROSION DAMAGE POTENTIAL WITH BRIDGE LENGTH AND EMBANKMENT HEIGHT.

There are many procedures available depending on the characteristics of the site and the quantity of data available. Procedures such as those developed by the Soil Conservation Service (Snider, 1972) and dimensionless hydrographs such as those developed by Craig (1970) or Commons (1942) are probably sufficiently accurate and reasonably simple to develop.

When data are available, a representative hydrograph could be selected from the data set perhaps corresponding to the flood of record. Hydrographs for the other floods selected can be computed by multiplying each ordinate by the ratio of the discharges, Q_i/Q_n, where Q_i is the peak discharge for which a hydrograph is described and Q_n is the peak discharge of the known hydrograph.

Natural Water-Surface Profiles

Various computer programs are available for computing water-surface elevations. For many applications the one-dimensional step-backwater programs such as those used by the Geological Survey (Shearman, 1976) or the Corps of Engineers (1972) are sufficiently accurate. When available, a measured water-surface profile, perhaps from surveyed floodmarks, should be used to calibrate the flood model selected to compute the water-surface profiles. Surveyed cross sections and field-selected Manning roughness coefficients are used to compute natural water-surface profiles, that is the water-surface profile which would exist if the bridge and embankment would not be built. A profile is computed for each of the previously selected peak discharges. The flood damage that would occur without the bridge can be computed using these data.

Flow Distribution

A determination of the distribution of flow in the approach cross sections or at the bridge centerline has often been included as a part of the hydraulic analysis. The information is used to assist the designer in locating and sizing the main channel bridge, relief bridges, culverts, and other drainage structures. Flow distribution can also assist in determining the need for and sizing spur dikes.

Special Considerations

Depending on the site, some special factors may have to be considered. These factors include debris, ice, farm dikes, changing land use, beaver dams, stream channelization, stream regulation, reservoirs, and levee construction. A comprehensive list of factors is not possible because every site is unique. However, a discussion of these special considerations should be made and allowance made for the risk and construction costs.

Alternative Bridges Considered in the Risk Analysis

At this point in the investigation, sufficient data are available to select a set of possible bridge configurations. In the example problems, a 10-ft range in embankment elevation and five to eight bridge lengths were selected. This was done to illustrate the results. In practice, however, the least costly project will be strongly related to the cost of the bridge and the degree to which it encroaches on the river channel.

Select a representative set of bridges and embankment heights which include bridges with low embankments (high cost of traffic interruption but little backwater damage) and short bridges with a high embankment (high cost of backwater damage but little traffic interruption). The shortest bridge selected should require channel protection to protect it from the 1-percent-chance flood. Construction costs which would be expected to reach a minimum at this point may actually increase because of the channel work. The risk to the structure at this point may be about the same as the annual construction cost for the channel work to protect the structure. Multiple bridge openings could help reduce backwater for sites where the flood plain is wide.

The cost of building each combination of embankment and bridge should be estimated. State transportation agencies have a great deal of experience and data which should be sufficiently accurate for these studies. There are other approaches, however. Unit cost procedures have been developed by Tseng (1975). Unit costs are available expressed in various units such as dollars per cubic yard of fill and dollars per square foot of deck. These cost data are used to estimate the construction cost of the bridge substructure and superstructure, the embankment, the pavement, appurtenant structures, and channel protection. Fill volumes are estimated assuming that the fill is trapezoidal in cross section (Tseng, 1975, p. 15-16).

Water-Surface Profiles

Shearman (1976) describes the Geological Survey step-backwater program E431. For each embankment-bridge combination, E431 could be used to develop a stage-discharge relation at the downstream side of the bridge and at the approach section, the upstream extent of backwater, and the water-surface profile for each flood. The most widely used backwater program may be HEC-2 by the Corps of Engineers (1972). None of the available programs appear to be completely applicable to every circumstance. Hence, on occasion, some engineering judgment may have to be applied.

The results of these computations are used in several sections of the bridge site investigation:

1. The computed water-surface profile, the natural water-surface profile, the land-use survey, and the depth-damage curves are used to compute damage caused by backwater.

2. The flood discharge hydrograph, the stage-discharge relation, and the minimum road bed elevation are used to compute duration of flow over the road (Tseng, 1975, p. 36-37).

3. The proportion of flow through the bridge and over the road are output from the E431 bridge routine. (A FHWA constraint requires that the bridge pass the design flood without inundating the highway.)

4. Flow distribution in the approach section can be computed from the conveyance distribution which is output from E431. Flow distribution is useful in locating relief drainage structures.

Risk Analysis

There are several procedures available for conducting the risk analysis leading to the selection of the design discharge. Design discharge is dependent on the bridge length and embankment height.

Procedures for conducting the risk analysis start in one of two ways:

1. Hold embankment height constant and vary bridge length.

2. Hold bridge length constant and vary embankment height (Tseng and others, 1975).

In the example problems the following procedure was used. A bridge of a certain deck length was assumed and the embankment height was varied in 1-ft or 2-ft intervals from approximately flood-plain level (the lowest practical elevation) up to a level that would be overtopped only under the most severe flood conditions. The hydraulics were computed for the bridge in combination with each embankment. Then another bridge length was selected and the computations repeated for each embankment height. This process was repeated until all selected bridges were analyzed.

In the hydraulic analysis (step backwater), water-surface profiles were computed for a series of discharges through each bridge and embankment combination. The same discharge series was used for each bridge and embankment combination.

The design discharge was calculated for each bridge and embankment combination by finding the discharge whose elevation upstream of the encroachment was equal to the minimum elevation of the encroachment.

The design discharge was used with the previously developed discharge hydrographs to compute the overtopping time--or the time that traffic would be interrupted. For each bridge and embankment, the design discharge was the point that the road over-flow began so that the discharge hydrographs were used to determine the time that the flow was in excess of the design discharge. The water-surface elevation hydrographs were used to estimate the maximum depth of flow over the embankment. This depth was used to estimate the erosion that would result from flow over the embankment.

Another approach used to calculate maximum depth of flow over the embankment involves the use of the water-surface elevation-discharge relation. A water-surface elevation-discharge relation may be known from measured data or can be computed at the downstream side of the bridge. This method is based on the assumption that the depth of flow over the road is approximately equal to the difference between the water-surface elevations of the various discharges and the design discharge of the bridge and embankment combination. The water-surface elevation for the design discharge and for each of the various discharges are determined from the water-surface elevation-discharge relation.

Figure 7 is a schematic showing the relation between design discharge, bridge length, and embankment height. For a constant bridge length, as embankment height increases, more water is forced to flow under the bridge. The elevation of the bridge is assumed to increase equally with embankment height. The relation shown in figure 7 is defined as the analysis proceeds.

The results of the analyses are summarized, as shown in table 1, for each bridge and embankment combination. The combination with the lowest TEPC (total expected project cost), which is the sum of the risk and the annual capital cost, is the theoretically least costly bridge crossing defined by those factors that were included in the analysis. The results of these studies are an input to the decision-making process which will select the final design discharge for the bridge crossing. Additional inputs include budgetary constraints, the need for emergency supply and evacuation routes, the need for emergency vehicle routes, and other considerations not included in the analysis.

As shown in the urban report (Appendix B), it may be useful to examine the effect of future development at the crossing to determine the impact of increased traffic loads or changing land use on the TEPC. This may be done as a sensitivity analyses, that is, multiples of the current conditions can be used in the analysis to estimate future development.

The relation between TEPC and exceedance probability of the design discharge can be relatively flat (fig. 1) near the theoretical minimum. For practical purposes within the accuracy of the results, the designer could choose from a range of design discharges for the bridge. This is discussed in more detail in Appendix B.

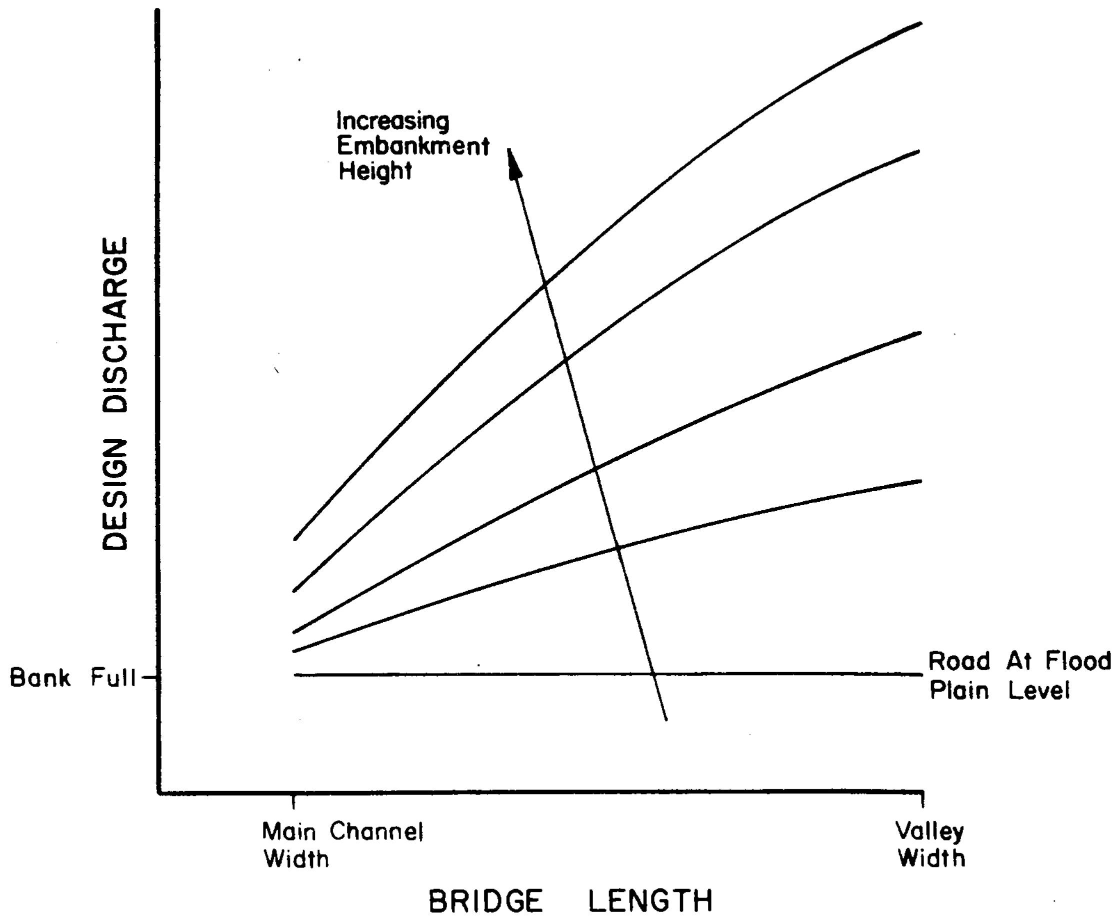

FIGURE 7. VARIATION OF DESIGN DISCHARGE WITH BRIDGE LENGTH AND EMBANKMENT HEIGHT.

Summary and Conclusions

The purpose of this section is to present the results of the risk analysis and to summarize the findings of the analysis. Describe factors if any, which qualify the design discharge selected. Actions taken as a result of the analysis might be noted. Recommendations for modifications to the structure, channel protection or other actions might be included.

SUMMARY

A procedure for conducting a bridge site investigation has been described in this report. The suggested report outline serves as the basis for organizing this manual. Each section has been annotated to describe techniques used and gives standard references and sources of information. The manual attempts to show how the data are collected and analyzed but does not describe, for example, the theory of water-surface profile computations.

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Appendix A

Hydraulic Design of a Bridge with Risk Analysis At Leaf River Near Collins, Mississippi

Example Study and Report

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At Leaf River Near Collins, Mississippi

INTRODUCTION

Acknowledgment

This report has been prepared as an example of a model bridge site study at a rural stream crossing. The work was accomplished under provisions of a Federal Highway Administration Research and Development contract. This study can be used by highway agencies as a model to conduct their own studies.

The Mississippi State Highway Department cooperated in the study and furnished the following data:

1. Plan-profile sheets for the proposed highway.

2. Channel-bottom profile in the vicinity of the site.

3. Detour route and daily cost of using detour for average daily traffic.

4. Construction cost estimates for the bridges and embankments used in the analysis.

All computations and base data used in this report are available.

All elevations in this report are referred to National Geodetic Vertical Datum of 1929 which is the datum used by the Mississippi State Highway Department in this project.

Purpose of this Study

The purpose of this study is to use risk analysis to determine the hydraulic design of a stream crossing which has the lowest possible TEPC (total expected project cost), that is, the optimum economic design.

Description of Project

U.S. Highway 84, an east-west highway, crosses the Leaf River 9-1/2 miles northeast of Collins. A new two-lane bridge and approach ramps will be constructed to replace the existing crossing. At present there is a 420-ft main-channel bridge, a 10 X 10-ft box culvert on the east flood plain, and two box culverts, a 6 X 6-ft and a triple 6 X 4-ft, on the west flood plain. The finished grade of the highway is at an elevation of 230 ft. State Highway 532 intersects U.S. Highway 84 about 500 ft west of the main channel and proceeds northwest across the flood plain for a distance of about a mile before it leaves the valley. State Highway 532 is constructed near flood-plain level. An old highway fill intersects the present fill about 500 ft east of the main channel, crosses the east flood-plain upstream from the present highway, and leaves the flood-plain about 700 ft upstream from U.S. Highway 84 at the east edge of the valley. This fill is approximately the same height as the present highway and has only one opening (10 X 10-ft box culvert); therefore, it partially blocks approaching flow and funnels it toward the main channel. The 10 X 10-ft box culvert goes through both the old and existing fills.

Existing highway alinement is shown in figure A-1, the U.S. Geological Survey topographic map of the vicinity. The proposed alinement of the new lane to be constructed at this time is parallel to the alinement of the existing bridge. The proposed bridge is about 135 ft downstream. The alinement is nearly a normal crossing of the flood plain and channel.

DATA COLLECTION

Flood-Plain Geometry

Three complete cross sections of the flood plain were surveyed approximately 4,500 ft downstream and 4,000 ft upstream of the centerline of the proposed highway crossing. Partial sections were surveyed upstream from and along the old road fill to define the geometry of the fill and the valley. A partial section of the east flood plain was surveyed downstream from the present highway. These data were used in the hydraulic analysis to compute the water-surface profiles. Their locations are shown in figure A-2 and the cross sections are shown in figures A-3A through A-3F.

Land Use

The valley is approximately half open and half wooded, with cultivated fields and pastureland occasionally reaching the banks of the river. In the immediate vicinity of U.S. Highway 84, about half of the west flood plain next to the river is cleared both upstream and downstream, whereas the east flood plain is covered with fairly large timber. Six houses of various value are on the west upstream flood plain within a mile of U.S. Highway 84. Land use was documented by aerial photographs obtained from the Department of Agriculture and ground photographs taken by the U.S. Geological Survey.

Development of the flood plain will not be appreciably affected by the Flood Insurance Program. Limited development is anticipated because the land has been owned in fairly large blocks for a number of years and the philosophy of the owners, plus the rural setting, would probably prohibit extensive development. The area is not within a reasonable commuting distance of an urban center.

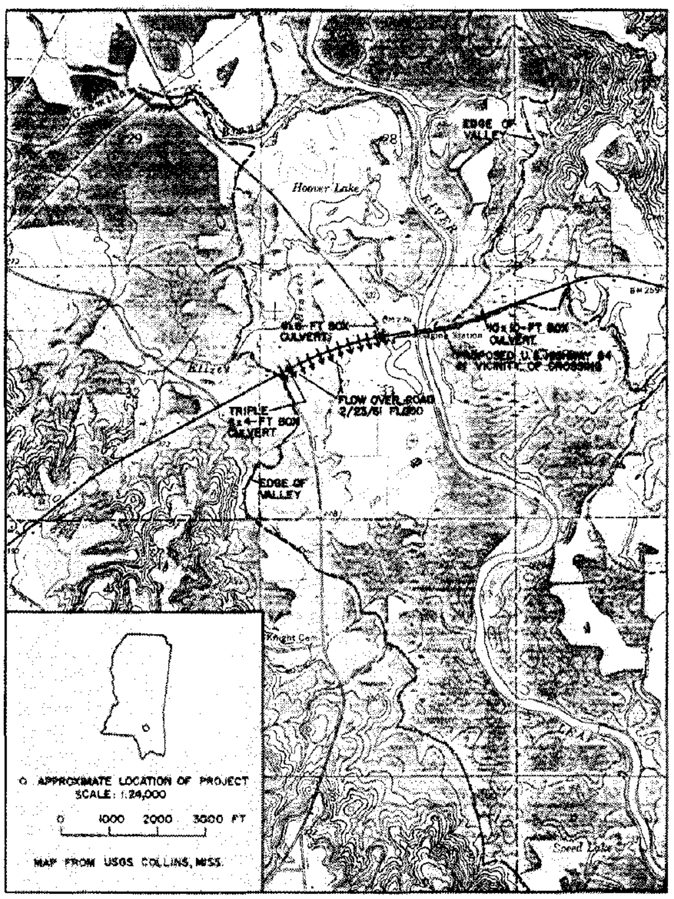

FIGURE A-1. GENERAL LOCATION MAP, LEAF RIVER AT U.S. HIGHWAY 84 NEAR COLLINS, MISS.

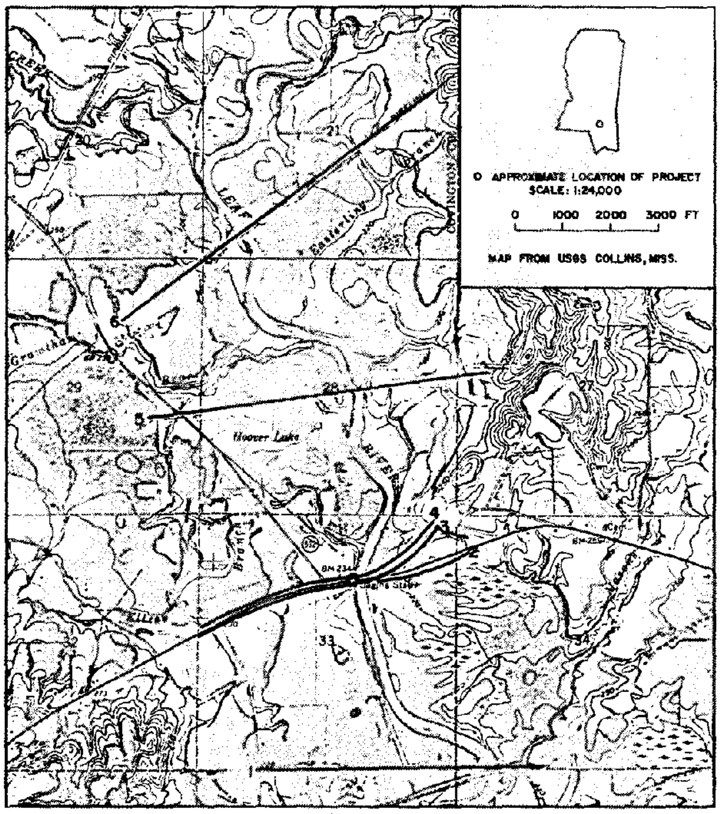

FIGURE A-2. LOCATION OF CROSS SECTIONS USED IN THE HYDRAULIC ANALYSIS, LEAF RIVER AT U.S. HIGHWAY 84 NEAR COLLINS, MISS.

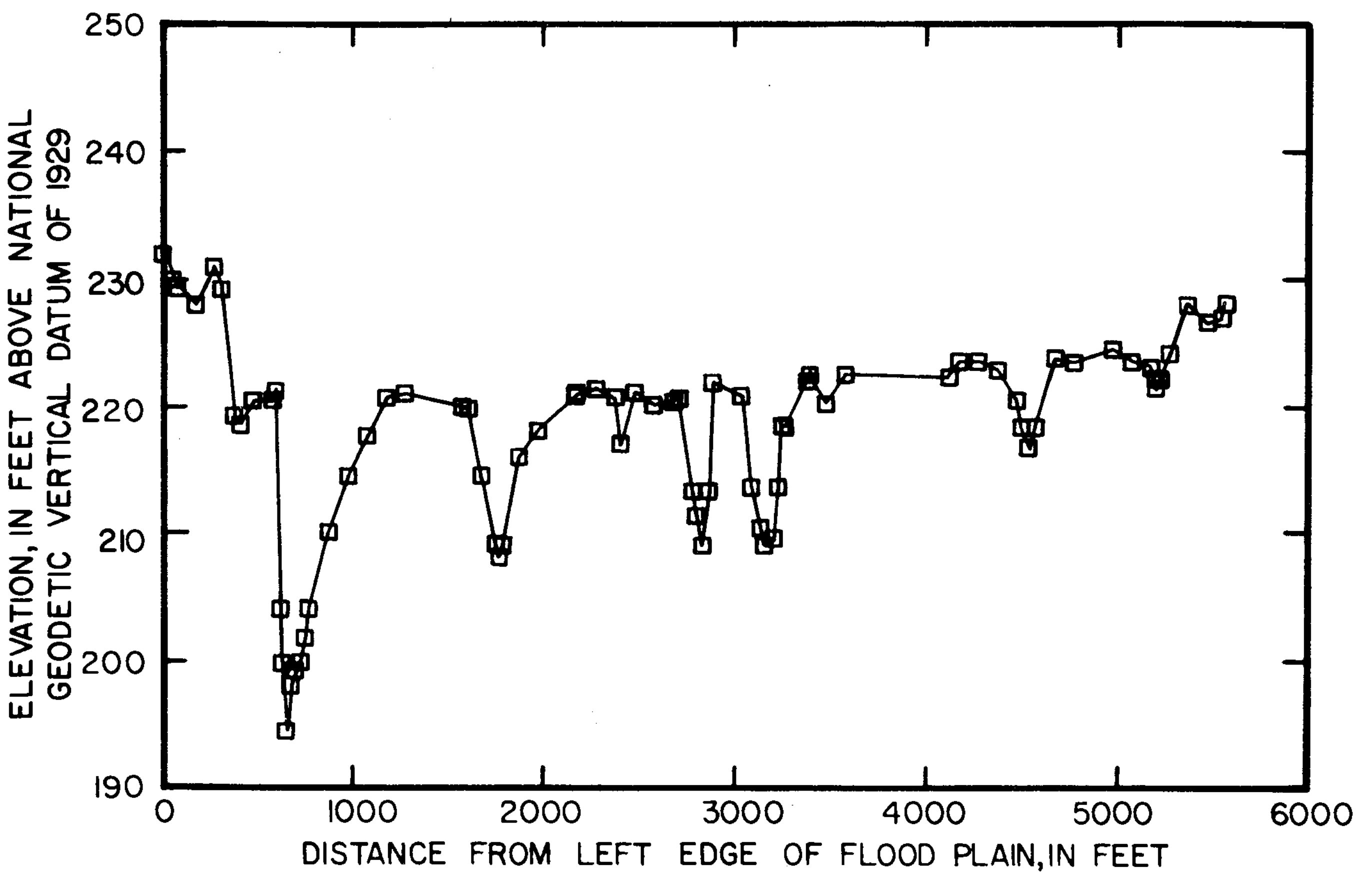

FIGURE A-3A. CROSS-SECTION 1 LOCATED DOWNSTREAM FROM THE PROPOSED U.S. HIGHWAY 84 BRIDGE (STATION 0 FT).

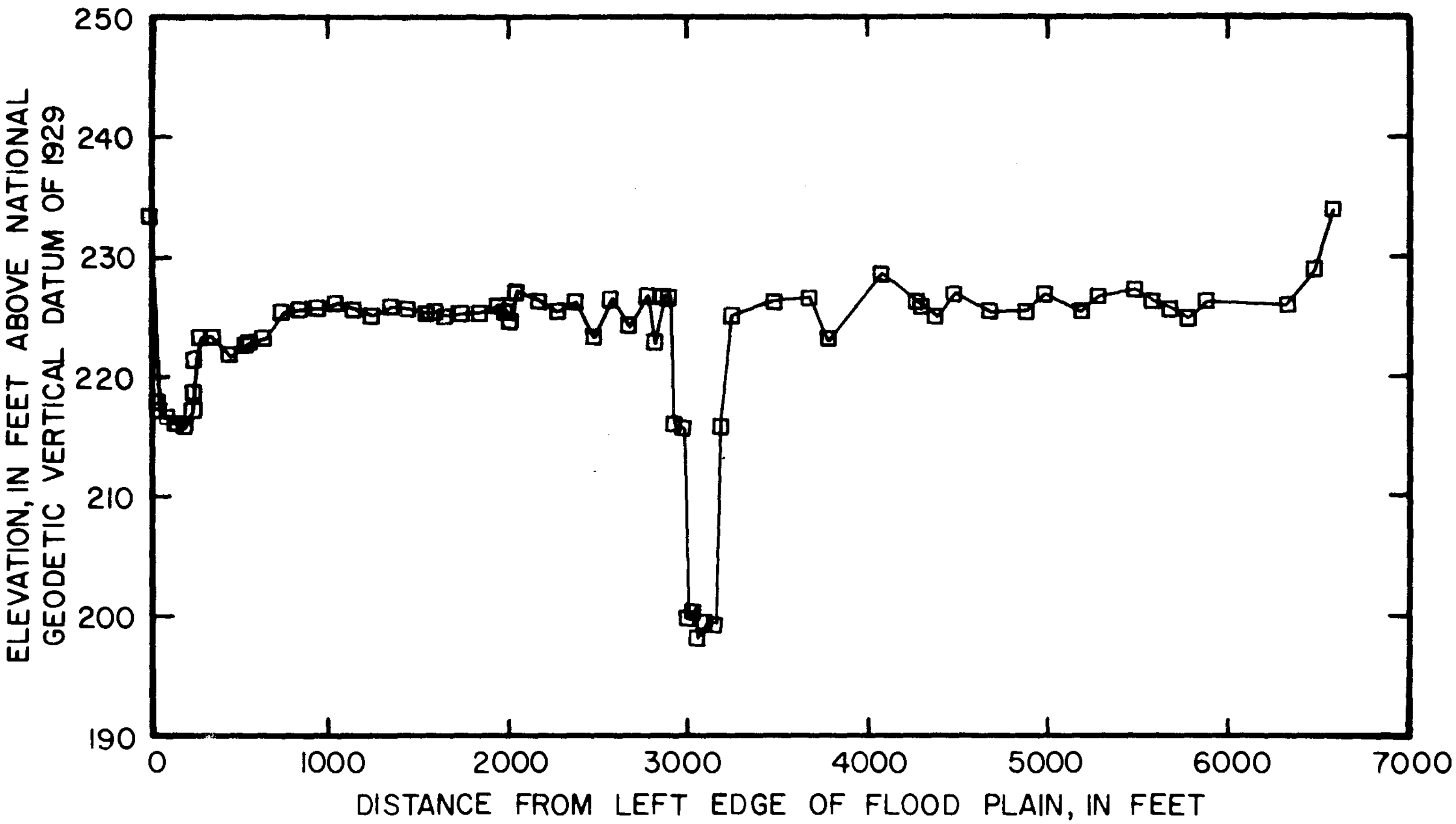

FIGURE A-3B. CROSS-SECTION 2 LOCATED IMMEDIATELY DOWNSTREAM FROM THE PROPOSED U.S. HIGHWAY 84 BRIDGE (STATION 4500 FT).

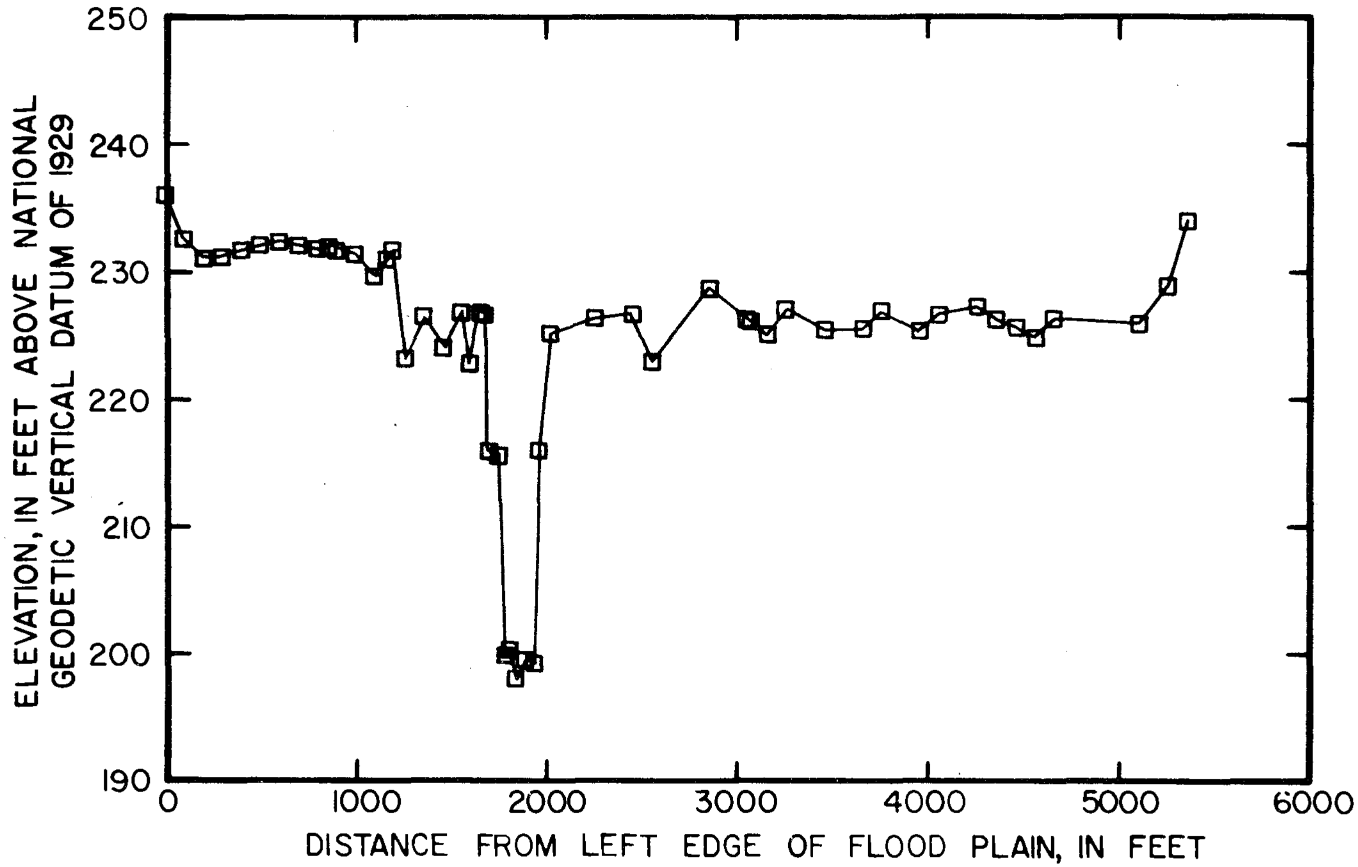

FIGURE A-3C. CROSS-SECTION 3 LOCATED UPSTREAM FROM THE PROPOSED U.S. HIGHWAY 84 BRIDGE (STATION 4700 FT).

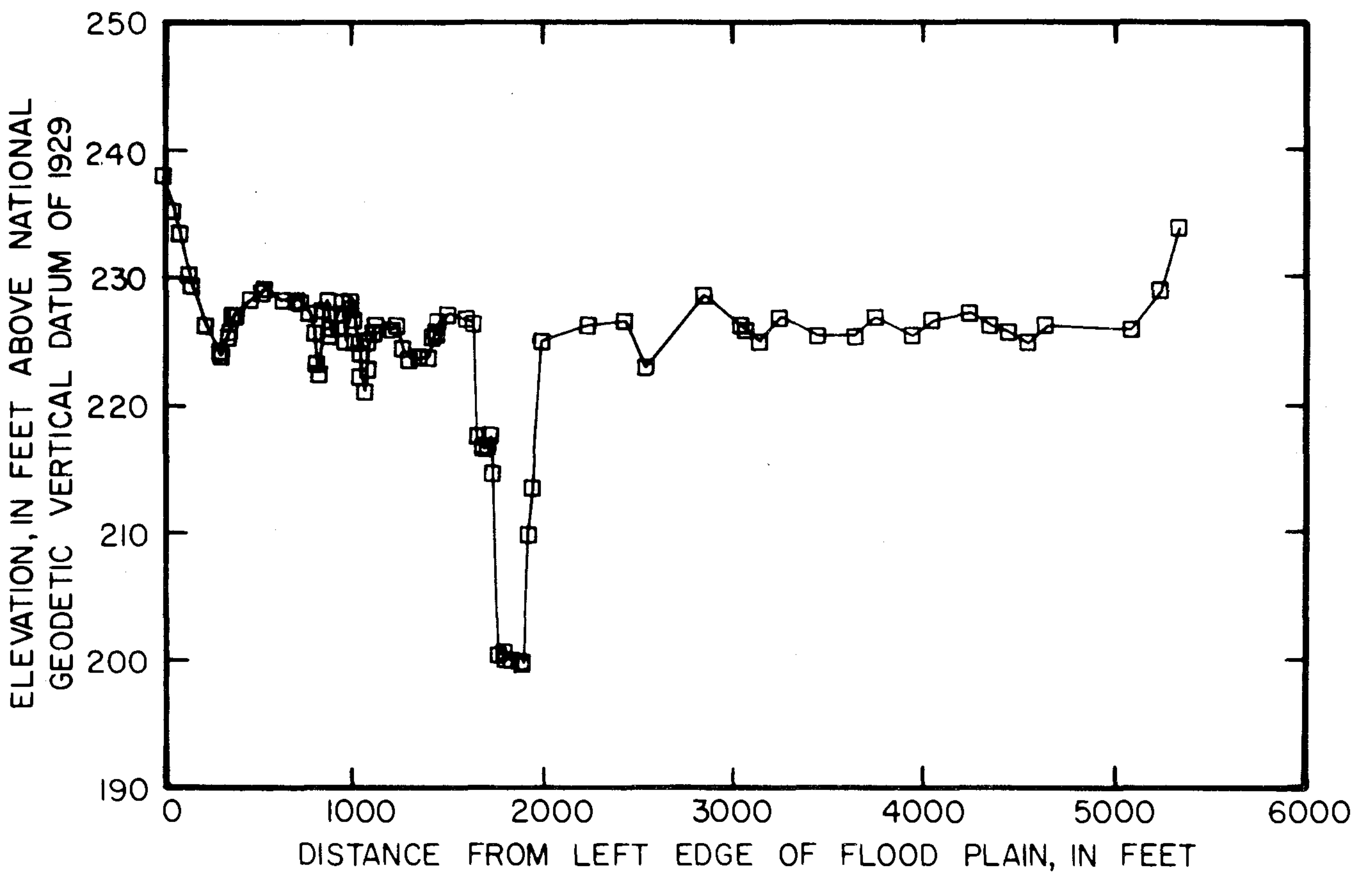

FIGURE A-3D. CROSS-SECTION 4 LOCATED UPSTREAM FROM THE PROPOSED U.S. HIGHWAY 84 BRIDGE (STATION 4800 FT).

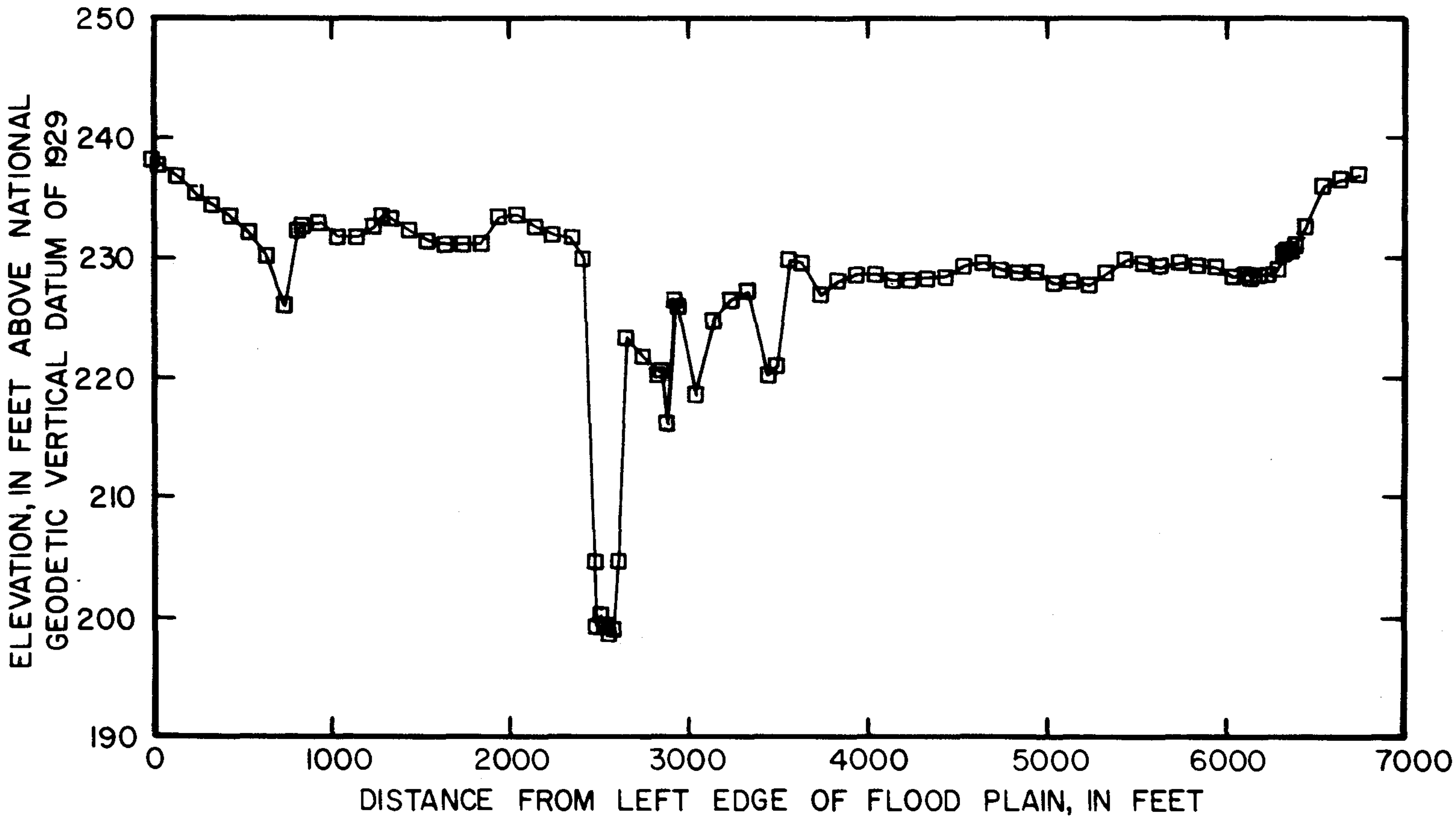

FIGURE A-3E. CROSS-SECTION 5 LOCATED UPSTREAM FROM THE PROPOSED U.S. HIGHWAY 84 BRIDGE (STATION 8500 FT).

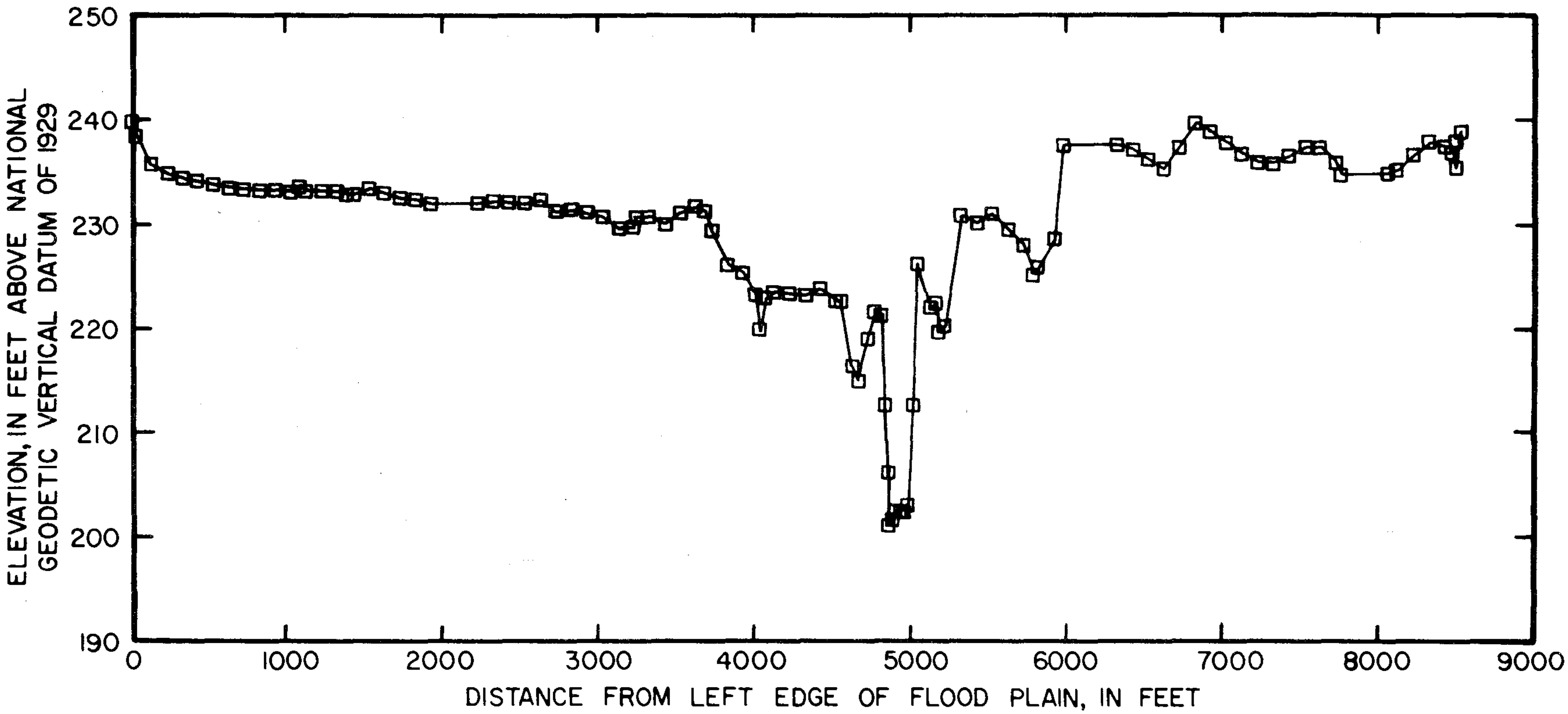

FIGURE A-3F. CROSS-SECTION 6 LOCATED UPSTREAM FROM THE PROPOSED U.S. HIGHWAY 84 BRIDGE (STATION 12,300 FT).

Hydrologic and Hydraulic Data

The U.S. Geological Survey has operated a continuous-record gaging station at this site since 1938. The largest flood during the period of record occurred on April 14, 1974, cresting at an elevation of 229.9 ft at the downstream side of the main-channel bridge. The peak discharge of this flood was 54,200 ft^3/s (cubic feet per second). The largest known flood at this site occurred in April 1856 and crested at an elevation of 230.5 ft according to local residents. The next largest flood occurred in April 1900 and reached an elevation of 229.5 ft. The peak discharges of these floods have been estimated as 56,000 and 50,000 ft^3/s, respectively.

The floods of February 1961 and April 1964 overtopped the present road grade (elev. 231 ft) on the west flood plain to maximum depths of 0.3 ft and 1.2 ft, respectively. Water was over the highway for about 36 hours during the 1974 flood. On the west flood plain, 2,900 ft west of the existing main-channel bridge, the April 1974 flood crested at an elevation of 232.1 ft upstream and 229.4 ft downstream according to surveyed flood marks by the U.S. Geological Survey. On the east flood plain, 1,300 ft east of the existing main-channel bridge, the April 1974 flood crested at an elevation of 228.3 ft on the upstream side of the embankment. The crest elevation of that flood on the east flood plain, 1,800 ft east of the main-channel bridge, was 228.0 ft on the downstream side of the embankment according to flood marks surveyed by the U.S. Geological Survey. The low upstream elevations on the east flood plain is caused by the old road fill just upstream partially blocking flood flows approaching U.S. Highway 84.

A discharge of 49,200 ft^3/s was measured April 13, 1974, at an elevation of 228.9 ft (1.0 ft below the crest). During the measurement 47,440 ft^3/s flowed through the bridge, 940 ft^3/s flowed over the highway and through two west flood plain culverts, and 800 ft^3/s flowed through one east flood plain culvert. Point velocities measured from the bridge were high in the main channel span, but were less than 4 ft/s in the approach spans. The maximum point velocity was 9.9 ft/s near the center of the main channel.

Geologic Data

Surface Geology

Surface geology in the vicinity is Catahoula sandstone which consists of perhaps 200 ft of clay, sandy clay, sand and gravel with some thin ferruginous (containing iron) layers. The beds of sand and gravel are capable of conveying large quantities of ground water.

Channel Morphology

The valley consists of a well-defined main channel with both a left and right overflow plain (fig. A-3E). The channel of Leaf River in the general vicinity of this crossing meanders very mildly at present, but old meander sloughs and lakes along the flood plain (figs. A-1, A-2) indicate that the meander of the river was more erratic at some former time. Study of aerial photographs and discussion with local residents indicate that the channel alinement has not changed appreciably in the period of record. The main channel for the most part is on the center third of the flood plain. The flood plain is relatively flat and slopes gently upward near the edges of the valley. At the stage of the 100-year flood, the average width of the flood plain is approximately 6,500 ft. The main channel is approximately 250 ft wide and 25 ft deep at bankfull stage.

Leaf River drains 752 mi^2 at this site and the valley slope is 2-1/2 ft/mi in the vicinity. The main channel carries about three-fourths of extreme flood discharges.

Leaf River at U.S. Highway 84 has been classified by Brice (1975) as "Stream Class Symbol 2C." This indicates a degree of sinuosity between 1.06 and 1.5 and a character of sinuosity of "single phase; wider at bends, chutes rare."

Soils Information

Considerable soils information has been collected at this crossing by the Mississippi State Highway Department (1975). The streambed is sand with pea gravel and the banks are formed of sand and clay strata. On top of the west bank, the top 20-ft stratum is sand. The 17-ft clay stratum thins to nothing at the edge of the low-water channel. The top 20-ft stratum is subject to sliding. The relative stability of the other banks in the vicinity is attributed to vegetation. Large trees line nearly all of the channel banks and the edge of the flood plain. The trees slow the velocities and their root systems reinforce the banks.

Scour History

The scour of the channel in the vicinity of the crossing during the April 1974 flood and the resulting sloughing of the west bank points to the need for special soils information to insure proper foundation for the new bridge. The channel has been stable since gaging records began in 1938 at this site except during the extreme floods of 1961 and 1974 during which times scour occurred. The 1961 flood scoured the channel to a depth of 6-10 ft below preflood level and the west bank was scoured about 8 ft. Between 1961 and 1974 the scour hole refilled. The 1974 flood scoured the channel to a depth of 10 to 15 ft at the bridge section and the scour extended several hundred feet upstream and downstream. Sloughing of the west bank following the flood pushed approach span piling out of line, endangering the present U.S. Highway 84 bridge. The scour hole which developed in 1974 has gradually filled during the 3 years since the flood.

Cost Data

Construction Costs

The Highway Department estimated construction costs for bridge lengths of 400, 480, 600, 720, and 800 feet for embankment heights varying from 227 to 235 feet in 1-foot increments. These costs are summarized in table A-1.

The increased costs for the 400-ft bridge are caused by the extensive channel protection work necessary to protect the west abutment. These works are not thought necessary for long spans which will be located away from the main channel. Embankment heights of 232 ft include the cost of two 100-ft-long elliptical spur dikes, and embankments of 233 to 235 ft include the costs of two 150-ft dikes.

Backwater Damage

There are six residences which could be damaged by backwater from the bridges. Velocities on the flood plain in the vicinity of the residences are low (less than 1 ft/s); therefore, damages from velocity were not considered. The data in tables A-2 and A-3 were used to compute damages to the structure and contents caused by backwater flooding. Backwater flood damage is the difference between the damage with the highway crossing in place and the natural condition (no highway).

Traffic Data

The present ADT (average daily traffic) was determined to be 2,222 vehicles. Traffic at this site was projected to grow at a compound rate of 3.5 percent per year. The equivalent uniform annual series for the average daily traffic is 3,375 vehicles. One suggested routing of this traffic when U.S. 84 is out of service is U.S. 49 from Collins to I-59 near Hattiesburg and I-59 near Hattiesburg to U.S. 84 near Laurel. The total length of this detour is 53 miles or 25 miles longer than the normal route. Analysis by the Highway Department, considering mixed vehicles, indicates that the average additional mileage cost per trip

Table A-1.--Construction costs in thousands of dollars for various bridge lengths and embankment heights, Leaf River at U.S. Highway 84 near Collins, Mississippi

Embankment heights (ft)	Bridge lengths (ft)				
	400	480	600	720	800
227	1480	1273	1343	1413	1458
228	1488	1281	1350	1420	1464
229	1500	1296	1365	1434	1479
230	1523	1317	1385	1453	1497
231	1550	1342	1409	1477	1520
232	1581	1375	1442	1508	1551
233	1613	1406	1472	1538	1580
234	1652	1443	1508	1572	1614
235	1697	1488	1548	1611	1652

Table A-2.--Flood depth versus structural damage, Leaf River at U.S. Highway 84 near Collins, Mississippi

Depth of flood above floor (ft)	Damage as percent of structure value
0	5
1	13
2	18
3	22
4	25
5	28
6	31
7	34
8	37
9	41
10	45
11	48

Table A-3.--Property which could be damaged due to backwater flooding, Leaf River at U.S. Highway 84 near Collins, Mississippi

Property owner	Current use	Initial damage elevation[1] (ft)	Location on flood plain station (ft)	Assessed value (dollars)
A	Residence	231.56	4800	20,000
B	Residence	230.69	4800	4,000
C	Residence	230.56	6400	12,000
D	Residence	228.83	6400	6,000
E	Residence	229.66	8200	80,000
F	Residence	231.11	8300	40,000

[1]First floor elevation of these residences.

utilizing the detour is $3.63 (14.5 cents/detour mile). The cost per trip of "lost time" was estimated to be $1.50 ($3 per hour). The increased chance of accidents was less than 2 percent according to Highway Department data and was not considered. Using the above figures, the total additional cost of moving 3,375 vehicles over the detour is $17,314 per day.

Embankment Repairs

When highway embankments are overtopped for sustained periods of time, they could be eroded. This erosion will result in traffic interruption and in the need for repairs to the embankment. Some experience is available in the State Highway Department concerning the duration of overtopping which will cause erosion to the extent that it is impassible and the length of time and cost needed to repair the damage. Figure A-4 defines the relation used to determine when a traffic-stopping washout will occur.

Damage is related to velocity and fill material. Velocities greater than 1 ft/s may begin to scour the sod shoulders of a well-sodded clay fill within 1 hour and velocities of 0.5 ft/s may begin to scour a poorly sodded sandy fill within 1 hour. Damage usually begins along the downstream shoulder but when a fill is superelevated on the upstream side, damage begins along the upstream shoulder.

Observations at a dozen or more sites which were overtopped 0.25 ft or less for periods of up to several days show no appreciable damage to well-sodded fills even when composed principally of sand. When flow approaches a depth of 0.5 ft for periods of only 1 hour, shoulder erosion is probable and flow at a depth of 0.5 ft for 18 hours may result in shoulder erosion averaging 4 ft wide by 1 ft deep plus loss of asphalt pavement averaging 1 ft wide along the entire length of the fill. Observations at one site which was overtopped 0.5 ft for 2 days showed shoulder erosion averaging 10 ft wide by 4 ft deep plus loss of asphalt pavement 6 ft wide.

Flow 2 ft deep for 1 hour may result in shoulder erosion 2 ft wide and 0.5 ft deep. Flow 5 ft deep for 1 hour may result in shoulder erosion 5 ft wide and 2 ft deep. Flow 3 ft deep for 2.5 days may erode the entire embankment.

Minor repairs would probably be made by Highway Department maintenance personnel using Department equipment to haul fill material procured nearby and paving it with asphalt from a nearby mix plant. Total washouts of several hundred feet of highway would probably be repaired by an emergency contract.

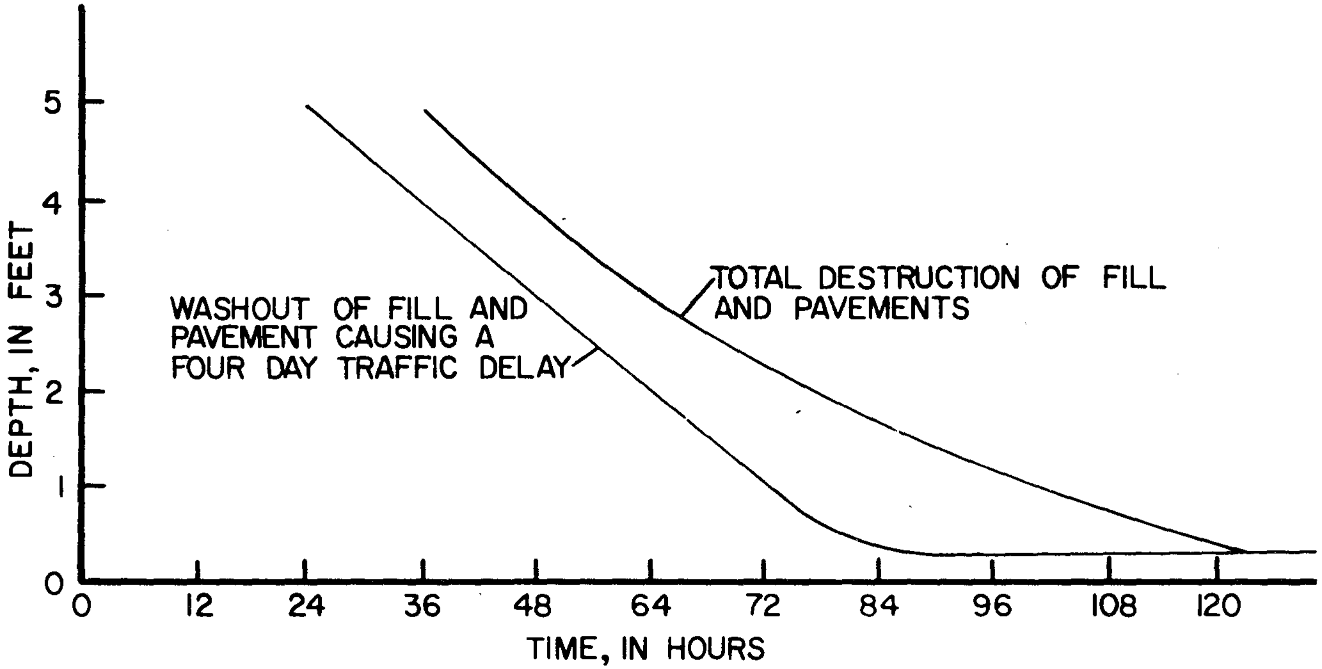

FIGURE A-4. ESTIMATED RELATION BETWEEN TIME AND DEPTH OF FLOW NECESSARY TO WASH OUT FILL AND PAVEMENT, LEAF RIVER AT U.S. 84 NEAR COLLINS, MISS.

Figure A-5 shows the relation between repair cost and the time and depth of overtopping for a 4-ft high (average) embankment. It is estimated that a 4-day traffic delay will occur if the embankment is breached to the extent that traffic cannot pass. These estimates are for a 100-ft length of highway with a 4-ft embankment. Adjustment coefficients (ratios of embankment costs) were used to adjust these curves for other embankment heights. This was accomplished by multiplying figures derived from figure A-5 by the ratio of the cost of the embankment being considered and the cost of a 4-ft embankment. These coefficients are shown in table A-4.

The relations shown in figure A-5 are based on the observations of a few situations described previously. The curve for an overtopping depth of 0.5 ft is defined by several observations. Other curves are defined by one or two points. Even though the flows described by the curves do occur, the predicted erosion very well may not occur. In constructing the curves, unit costs of $2 per cubic yard for fill and $14.40 per square yard for asphalt pavement were used.

DATA ANALYSIS

Magnitude and Frequency of Floods

A discharge-frequency curve for this site (fig. A-6) is based on the annual peak discharges for the period 1938-76 and the historical peak discharges of April 1856 and April 1900. This log-Pearson type 3 frequency curve was developed using procedures outlined by the United States Water Resources Council (1977). According to figure A-6, the 2-percent-chance flood at this site is 47,200 ft^3/s, and the 1-percent-chance flood is 56,100 ft^3/s. Floods of 1900 (56,000 ft^3/s), 1974 (54,200 ft^3/s), and 1961 (48,500 ft^3/s) had exceedance probabilities of about 1, 1.25, and 2 percent.

Hydrographs

The shape of the 1961 flood hydrograph was determined to be typical after comparison with other extreme floods and was used as a model for estimating hydrographs (figs. A-7 and A-8) for the flood peaks (table A-5) which were used in determining the depths and periods of overtopping (and resulting damages) of the various bridges and fill.

Natural Water-Surface Profiles

A stage-discharge relation (fig. A-9) was developed using numerous current-meter discharge measurements obtained at this site during the period 1938-76. According to figure A-9, the elevation (at the downstream

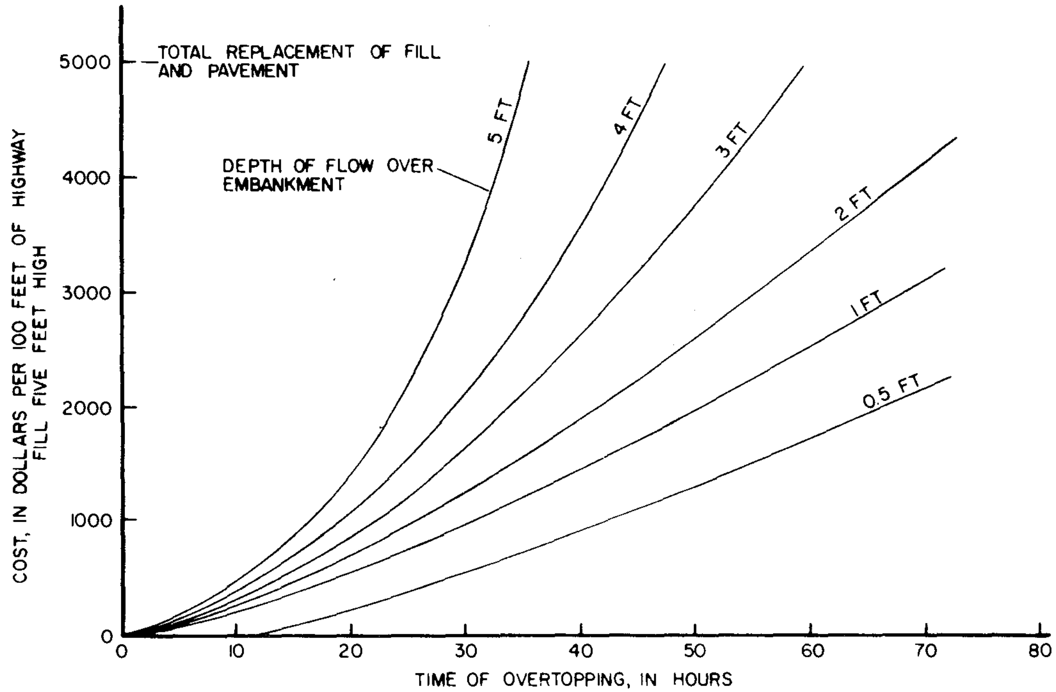

FIGURE A-5. COST OF REPAIRING A 24-FOOT-WIDE ROAD AND FILL 5 FT HIGH AS A FUNCTION OF DEPTH OF FLOW OVER THE EMBANKMENT AND TIME.

Table A-4.--Ratios of embankment costs to 4-ft embankment costs, Leaf River at U.S. Highway 84 near Collins, Mississippi

Embankment elevation (ft)	Adjustment coefficient
227	0.57
228	0.62
229	0.72
230	0.85
231	1.00
232	1.19
233	1.36
234	1.58
235	1.83

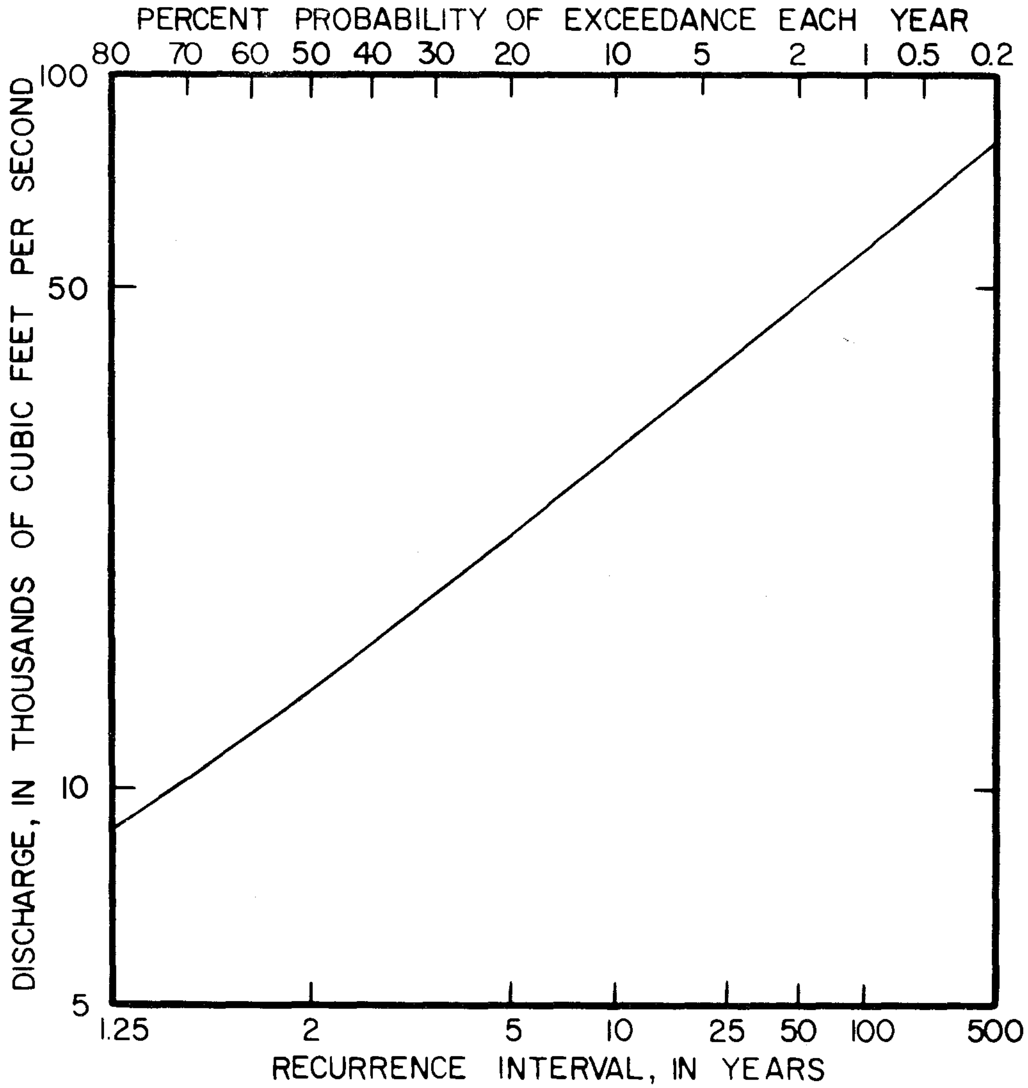

FIGURE A-6. MAGNITUDE AND FREQUENCY OF ANNUAL FLOODS, LEAF RIVER AT U.S. HIGHWAY 84 NEAR COLLINS, MISS.

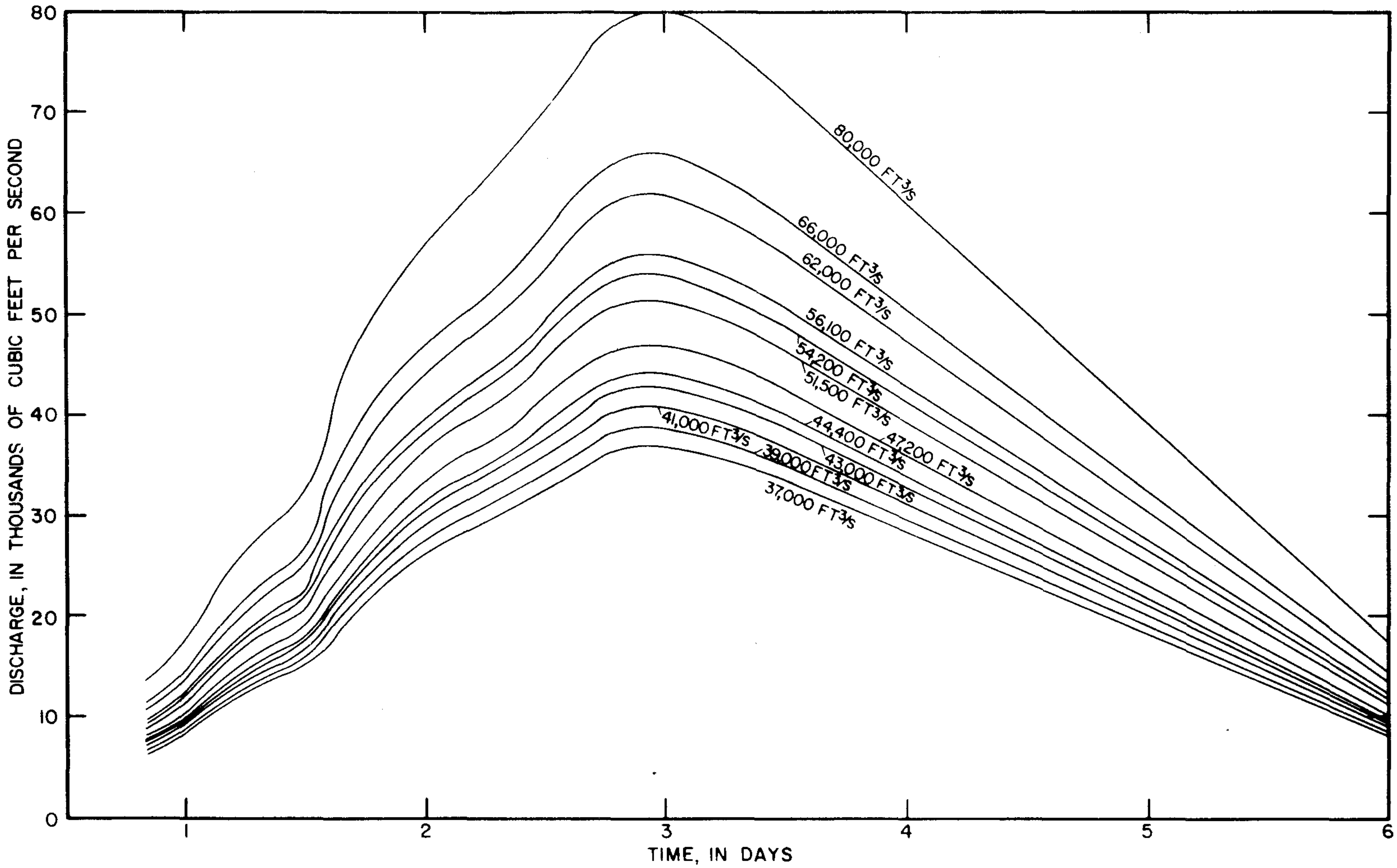

FIGURE A-7. FLOOD HYDROGRAPHS, LEAF RIVER NEAR COLLINS, MISS.

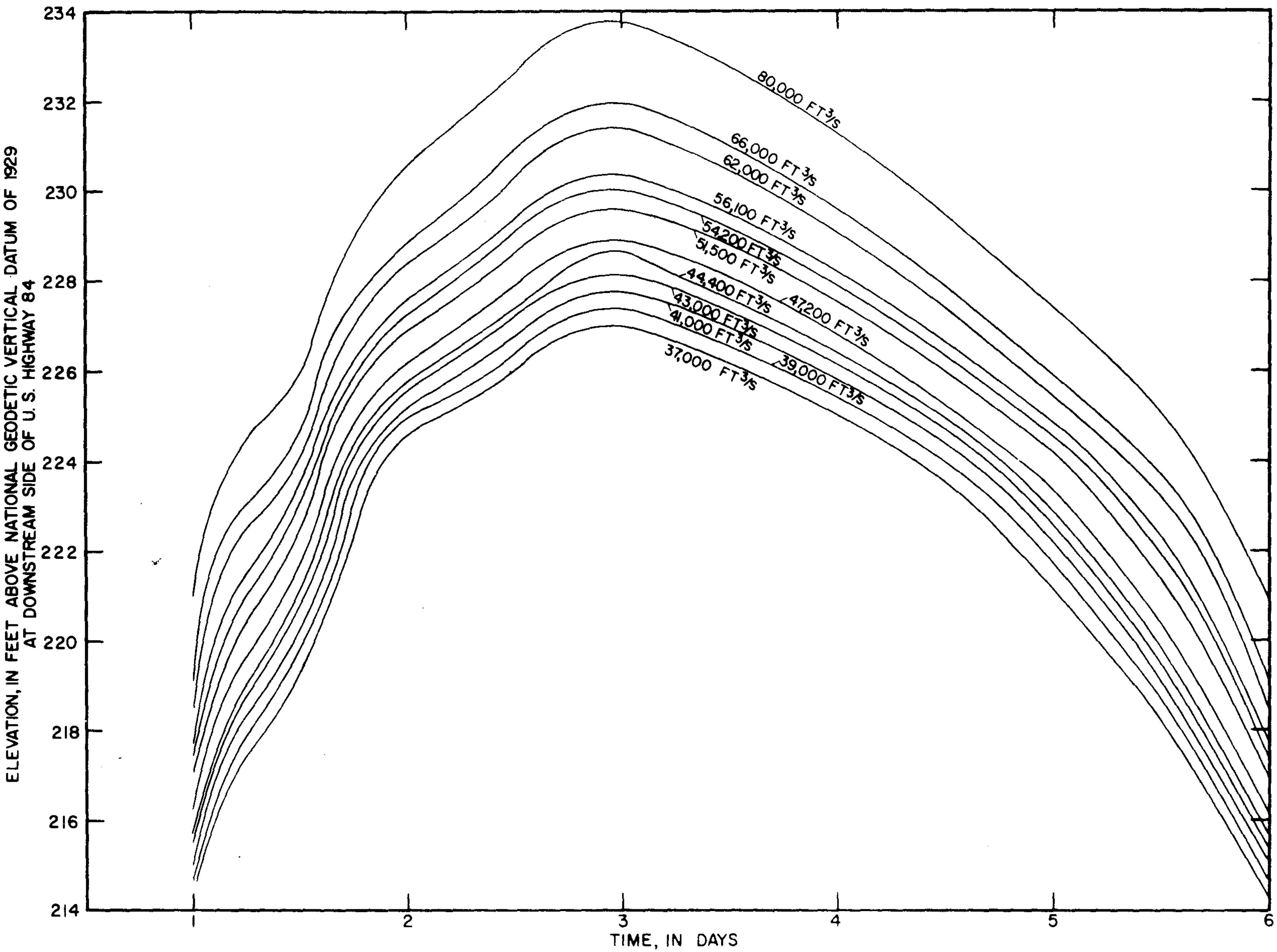

FIGURE A-8. FLOOD HYDROGRAPHS, LEAF RIVER AT U.S. HIGHWAY 84 NEAR COLLINS, MISS.

Table A-5.--Flood peaks used in determining depths and periods of overtopping, Leaf River at U.S. Highway 84 near Collins, Mississippi

Flood discharge (ft^3/s)	Exceedance probability (percent)
37,000 (bankfull)	4.8
39,000	4.0
41,000	3.3
43,000	2.9
44,400	2.5
47,200	2.0
51,500	1.4
54,200	1.2
56,100	1.0
62,000	0.7
66,000	0.5
80,000	0.2

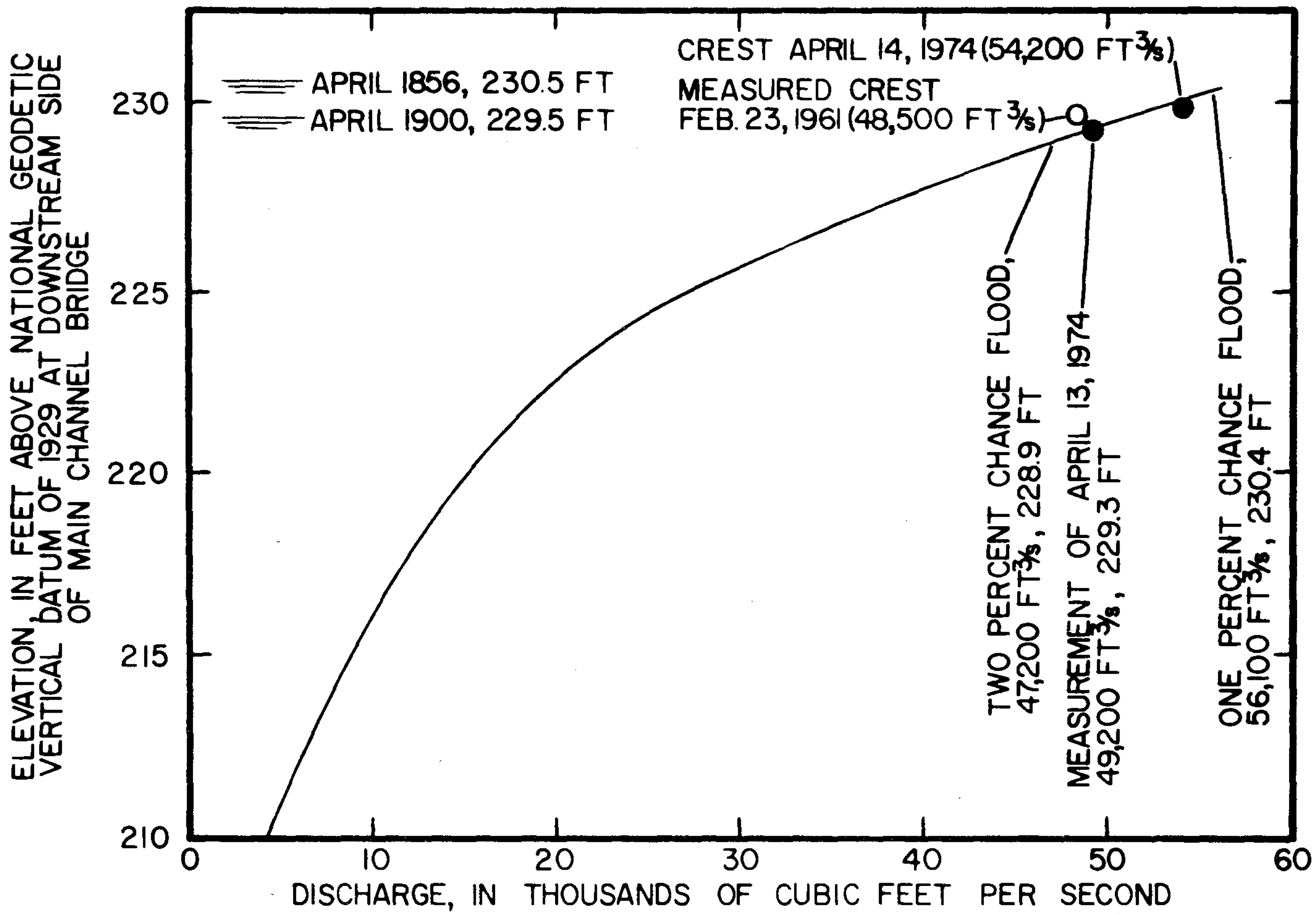

FIGURE A-9. STAGE-DISCHARGE RELATION, LEAF RIVER AT U.S. HIGHWAY 84 NEAR COLLINS, MISS.

side of the main-channel bridge) corresponding to the 50-year discharge (47,200 ft^3/s) is 228.9 ft and the elevation corresponding to the 100-year discharge (56,100 ft^3/s) is 230.4 ft. At bankfull stage (227 ft) the discharge is 37,000 ft^3/s, all of which is confined to the main channel. This discharge has an exceedance probability of 0.048 according to figure A-6.

The longitudinal water-surface profile (before highway construction) was computed by the step-backwater method using valley cross sections located 4,500 ft downstream, at the centerline of the proposed crossing, and 4,000 and 7,800 ft upstream. Profiles for the bankfull flood (37,000 ft^3/s), the 0.2-percent-chance flood (80,000 ft^3/s), and the flood of April 14, 1974, (54,200 ft^3/s) are shown in figure A-10 along with the recovered flood profile of April 1974 (with the highway in place). The longitudinal profiles were computed using the U.S. Geological Survey step-backwater program (Shearman, 1976).

Flow Distribution

The calculated flow distribution was based on the relative conveyance of several subsections of the centerline cross section. The distribution is made by the step-backwater method computer program using Manning's equation. The results are shown in figure A-11.

During the 2-percent-chance flood, 37,700 ft^3/s approaches in the main channel, 6,600 ft^3/s is distributed on the west flood plain, and 2,900 ft^3/s on the east flood plain. During the 1-percent-chance flood, 38,600 ft^3/s approaches in the main channel, 12,900 ft^3/s is distributed on the west flood plain, and 4,600 ft^3/s on the east flood plain. According to these results approximately 70 percent of the 1-percent-chance flood is confined to the main channel.

On April 13, 1974, a current-meter measurement of 49,200 ft^3/s was obtained at an elevation of 228.9 ft (1.0 ft below the crest). The measured distribution of this discharge through the bridge, three culverts, and over the highway is shown in figure A-11. The measurement shows 47,440 ft^3/s through the bridge opening, 940 ft^3/s over the highway and through the two west flood plain culverts, and 800 ft^3/s through the one east flood plain culvert. Crest elevations of April 14, 1974, near both edges of the valley upstream and downstream from the bridge are shown on the valley cross section along the proposed highway alinement (fig. A-11). According to these elevations, the difference in water-surface elevations across the embankment on the west flood plain near Station 48300 was 2.7 ft and on the east flood plain near Station 53000 was 0.8-ft.

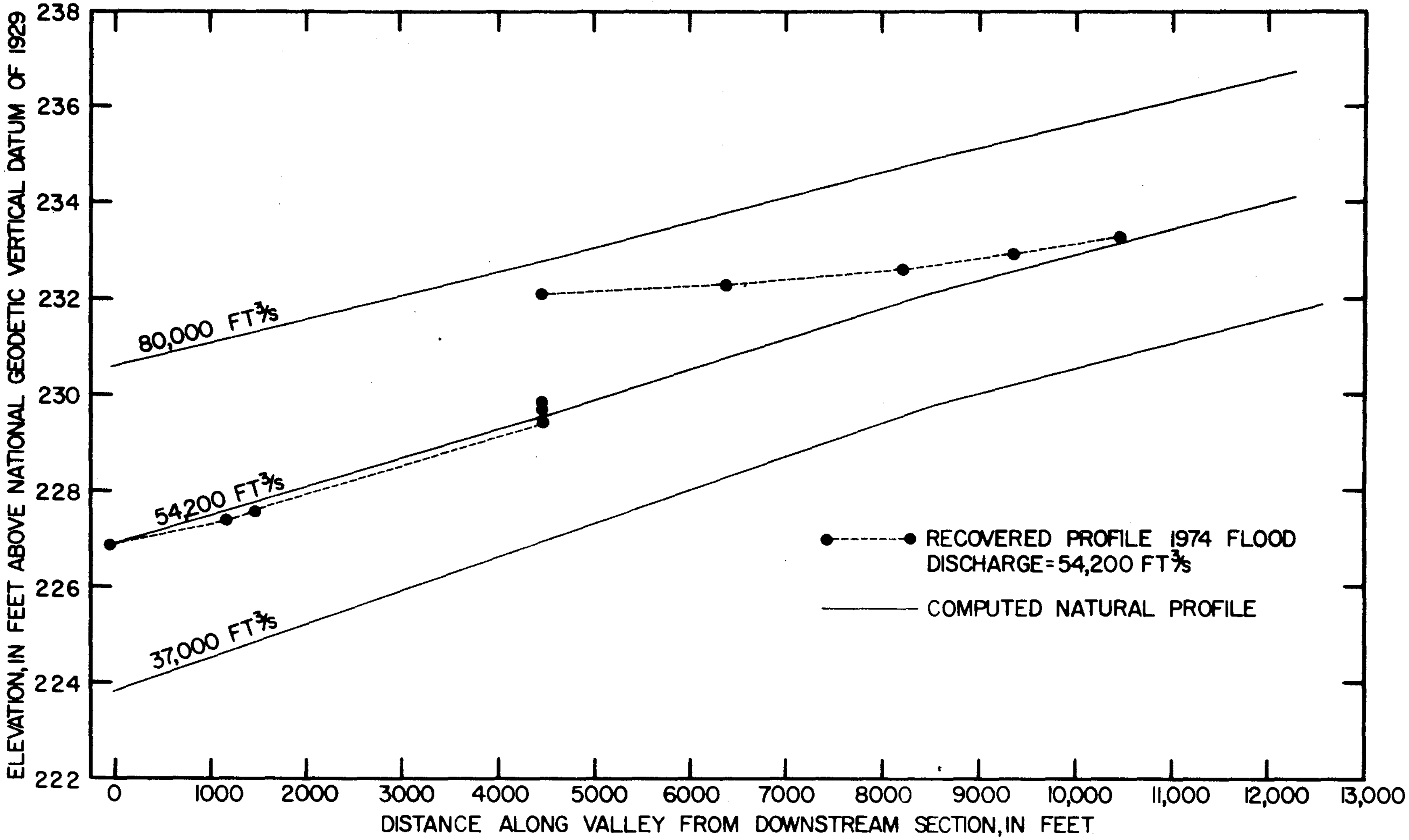

FIGURE A-10. TYPICAL COMPUTED NATURAL WATER-SURFACE PROFILES.

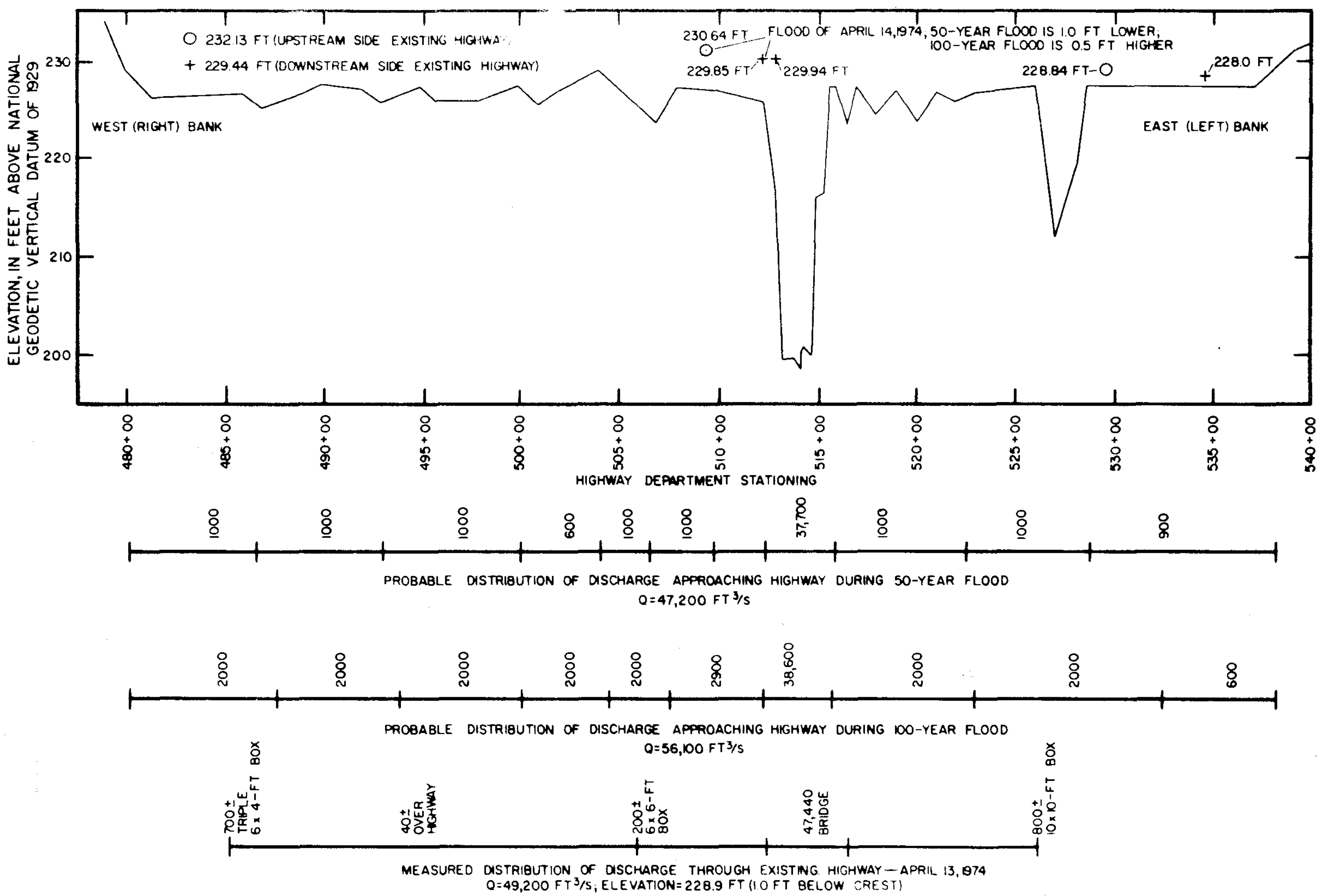

FIGURE A-11. CROSS SECTION AND PROBABLE DISTRIBUTION OF FLOW, LEAF RIVER AT U.S. HIGHWAY 84 NEAR COLLINS, MISS.

Special Considerations

Drift

Drift at this site should be minor according to observations of the U.S. Geological Survey near the crest of 1961 and 1974 floods, during which time little or no drift was observed.

Soils

Sloughing of the west bank following the flood of April 1974 pushed approach span piling out of line, endangering the present bridge. Because of this unstable bank, 160-ft approach spans were considered necessary to span the channel banks without channel protection. With the addition of channel protection, the minimum bridge length considered by the Highway Department is 400 ft (three spans--120 ft, 160 ft, and 120 ft.)

Spur Dikes

Spur dikes are not needed at this site for embankment heights below 232 ft because of the low percentage of flow on the flood plains for floods below that elevation. Analyses of higher embankments using the criteria of Bradley (1973) indicated that 100-ft spur dikes are needed at both abutments for embankment heights of 232 ft and that 150-ft dikes are needed for embankment heights of 233 to 235 ft. The costs of these dikes were estimated by the Highway Department and are incorporated in the construction costs.

Channel Protection

Channel protection for the west abutment is necessary for the 400-ft-long bridge because it does not span the area subject to sloughing. The cost of the protection for this arrangement is $250,000. The longer bridges are arranged to span the difficult soil and foundation conditions on the west bank.

Alternative Bridges Considered in the Risk Analysis

Five bridges were analyzed in the risk analysis. The low chord in each case was set so that it was not submerged for the hydraulic conditions considered in this report. The five combinations include:

1. 400-ft bridge (three spans--120 ft, 160 ft, and 120 ft).
2. 480-ft bridge (three spans at 160 ft each).
3. 600-ft bridge (three spans at 40 ft each and three spans at 160 ft each).

4. 720-ft bridge (six spans at 40 ft each and three spans at 160 ft each).

5. 800-ft bridge (eight spans at 40 ft each and three spans at 160 ft each).

Soil conditions and the size of the main channel prevent consideration of a bridge shorter than 400 ft. Nine approach embankments were considered for each bridge. The elevation was varied from 227 to 235 ft in 1-foot increments. Therefore, 45 possible combinations of bridges and embankments were considered at this site.

Water-Surface Elevation Profiles

High-water elevations were obtained after the 1974 flood from high-water marks around the existing embankment and at the gage. Residents of the area were of help in obtaining high-water elevations near the cross sections 4,500 ft downstream and 4,000 and 7,800 ft upstream of the proposed highway centerline. The 1974 flood profile was computed using the step-backwater program (Shearman, 1976) for the existing U.S. 84 bridge and the 1974 peak discharge. The roughness coefficients used in computing the natural profile were used unchanged. The measured profile, determined from high-water mark elevations, was compared to the computed profile. When the computations were adjusted using methods for wide flood plains (Schneider and others, 1976), the results shown in figure A-12 were obtained.

The additional acreage inundated by backwater during the April 1974 flood was small because of the valley shape. In other words, based on backwater computations the extra inundation width at the section of maximum backwaters averaged 75 ft on each side of the flood plain. This width tapered to zero at approximately 6,000 ft from the bridge opening. It is approximated as:

$$\frac{6{,}000 \times \frac{150}{2}}{43{,}560} = 10 \text{ acres.}$$

With this satisfactory agreement for the 1974 flood, the hydraulics were computed using each of the 45 bridge and embankment combinations described previously. Typical computed profiles are shown in figure A-13.

RISK ANALYSIS

A flood passing through this bridge may cause several types of damage. These include traffic delay owing to flow over the embankment and embankment washout, damage owing to increased backwater and the cost

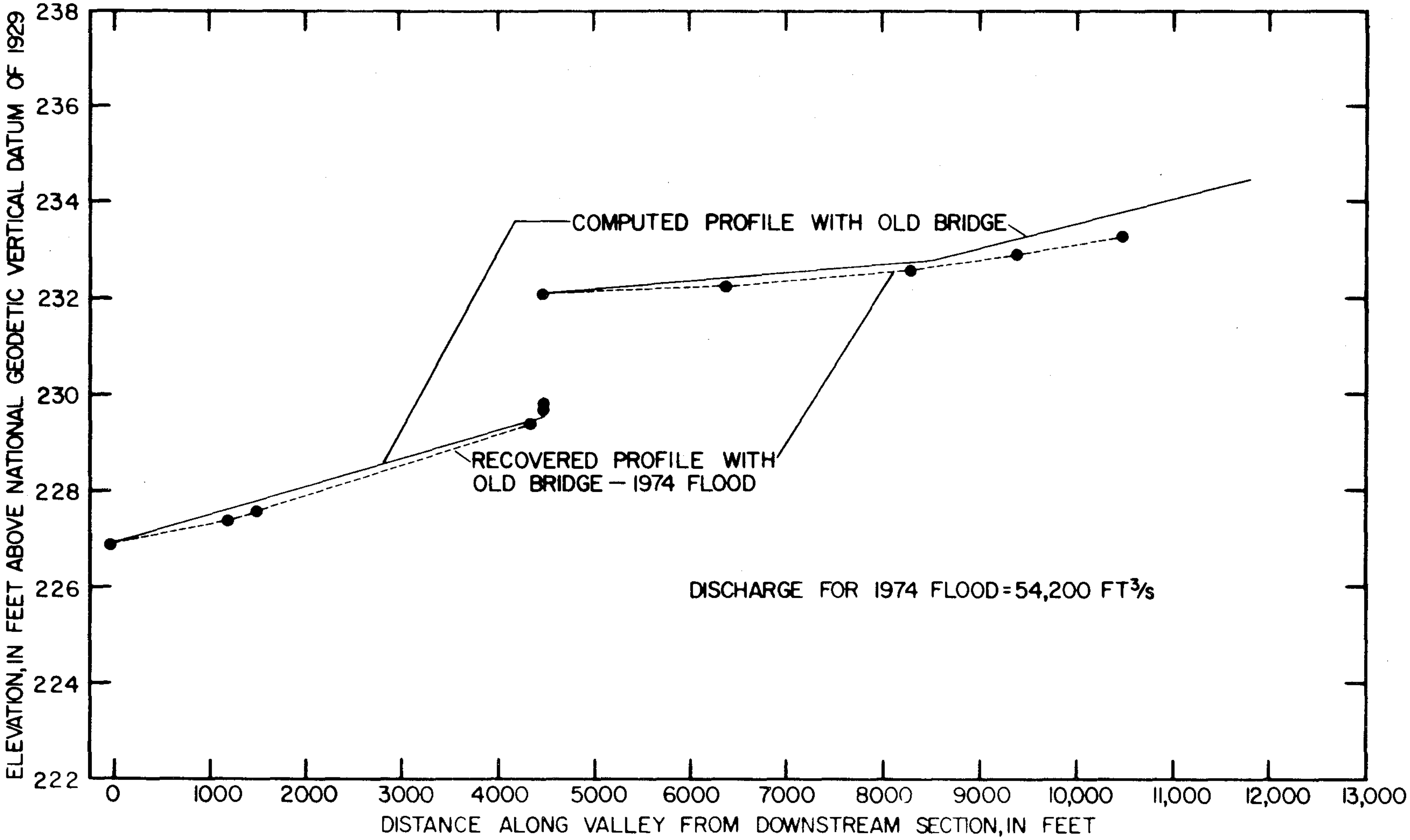

FIGURE A-12. COMPARISON BETWEEN THE COMPUTED AND RECOVERED PROFILES FOR 1974 FLOOD.

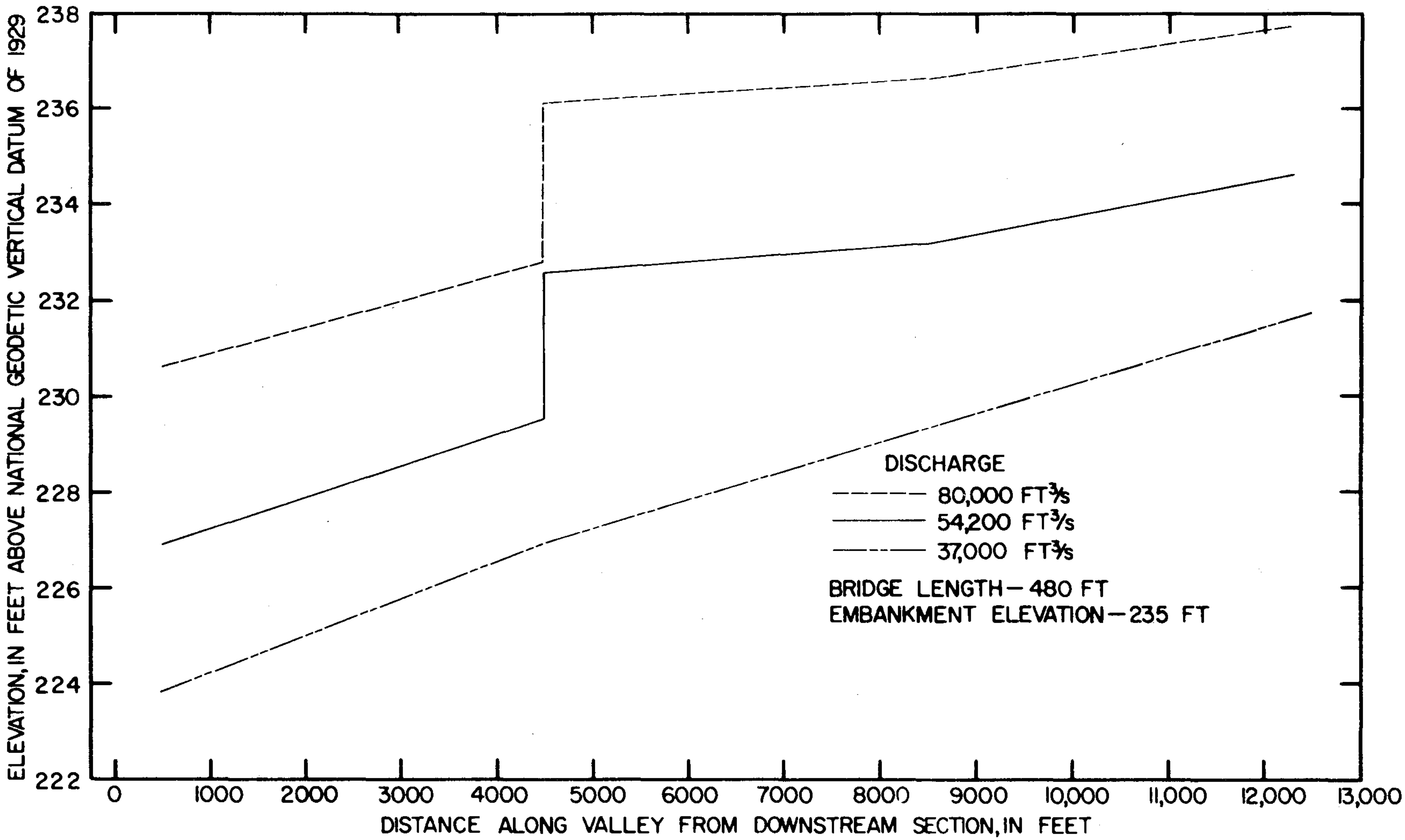

FIGURE A-13. TYPICAL COMPUTED WATER-SURFACE PROFILES WITH THE BRIDGE.

of embankment repairs. The bridge is assumed not to be damaged during any event since the piers and foundations are set at sufficient depth so that they will not be undermined by scour and since debris is not expected to be a problem at this site.

After the flood profiles for the natural and constricted conditions were computed, the resulting stages, times, and depths were used in the damage relations defined previously to compute damages. Table A-6 shows typical results which occur for a bridge 480 ft long and an embankment elevation of 231 ft.

Damage due to each flood peak was computed for each combination. An example data summary is shown in table A-6. The risk or expected damage was computed by averaging the damage over two adjacent discharges, multiplying by the increment of probability, and summing over all possible combinations. The annual construction costs were computed assuming a 5-percent interest rate and a 25-year project life. The resulting capital recovery factor is 0.07095. (Multiply the total construction cost shown in table A-1 by the capital recovery factor to get the annual cost.)

The relation between construction costs and bridge length for each embankment height is shown in figure A-14. The 480-ft bridge with an embankment elevation of 227 ft has the smallest construction cost. The need for channel protection increases the construction cost of shorter bridges.

Annual construction cost and risk are compared to bridge length in figure A-15 and embankment elevation in figure A-16. Low risk values are due in part to sparse development in the flood plain. The risk is larger for shorter bridges and for the lowest embankment elevations. The channel protection could be lost during the larger floods and the traffic interruption increases the risk due to flow over the road. In any case, the risk is very small at this site being less than $8,500 for the worst case.

The annual construction cost and the risk are summed and plotted in figure A-17 as a function of embankment height and bridge length. The addition of risk associated with the shortest bridge and lowest embankment has little effect as it is only about 3 percent of the annual construction costs at this site.

The relation in figure A-18 was derived by computing the maximum discharge that would pass through a given bridge without overtopping the embankment. This value is defined as the design discharge for that bridge and embankment. In addition, the mean velocity of the flow through the bridge is plotted in this figure. Since the valley is

Table A-6--Summary of risk analysis

Site: Leaf River near Collins, Miss.

Design Discharge: 49,000 ft^3/s

Embankment Height: 231 feet

Bridge Deck Length: 480 feet

Estimated Capital Cost: $1,342,000

Estimated Service Life: 25 years

Interest Rate: 5 percent

Annual Capital Cost: $95,215

Flood ft^3/s	Flood probability	Maximum depth over roadway, feet	Duration of flow over the road, hours	Time road is closed for repairs, hours	Cost in dollars: Traffic interruption	Cost in dollars: Backwater damage	Cost in dollars: Overtopping damage to embankment	Cost in dollars: Other damage	Total damage, dollars	Average damage for interval, dollars	Probability interval, ΔP	Risk, dollars
37,000	0.048					0			0	100	0.008	1
39,000	.040					200			200	750	.007	5
41,000	.033					1,300			1,300	1,850	.004	7
43,000	.029					2,400			2,400	3,200	.004	13
44,400	.025					4,000			4,000	4,100	.005	20
47,200	.020					4,200			4,200	9,579	.006	57
51,500	.014		14		6,649	6,300	2,000		14,949	27,080	.002	54
54,200	.012		22		10,449	9,762	19,000		39,211	46,426	.002	93
56,100	.010		27		12,824	9,318	31,500		53,642	75,300	.0033	248
62,000	.00667		40		18,998	7,901	70,000		96,959	115,378	.00167	194
66,000	.0050		48		22,798	0	112,000		134,798	193,845	.003	582
80,000	.002		68	96	77,893	0	175,000		252,893	252,893	.002	506
Ultimate					77,893	0	175,000		252,893			

Total Annual Risk	$ 1,780
Annual Capital Cost	$95,215
Total Expected Project Cost	$96,995

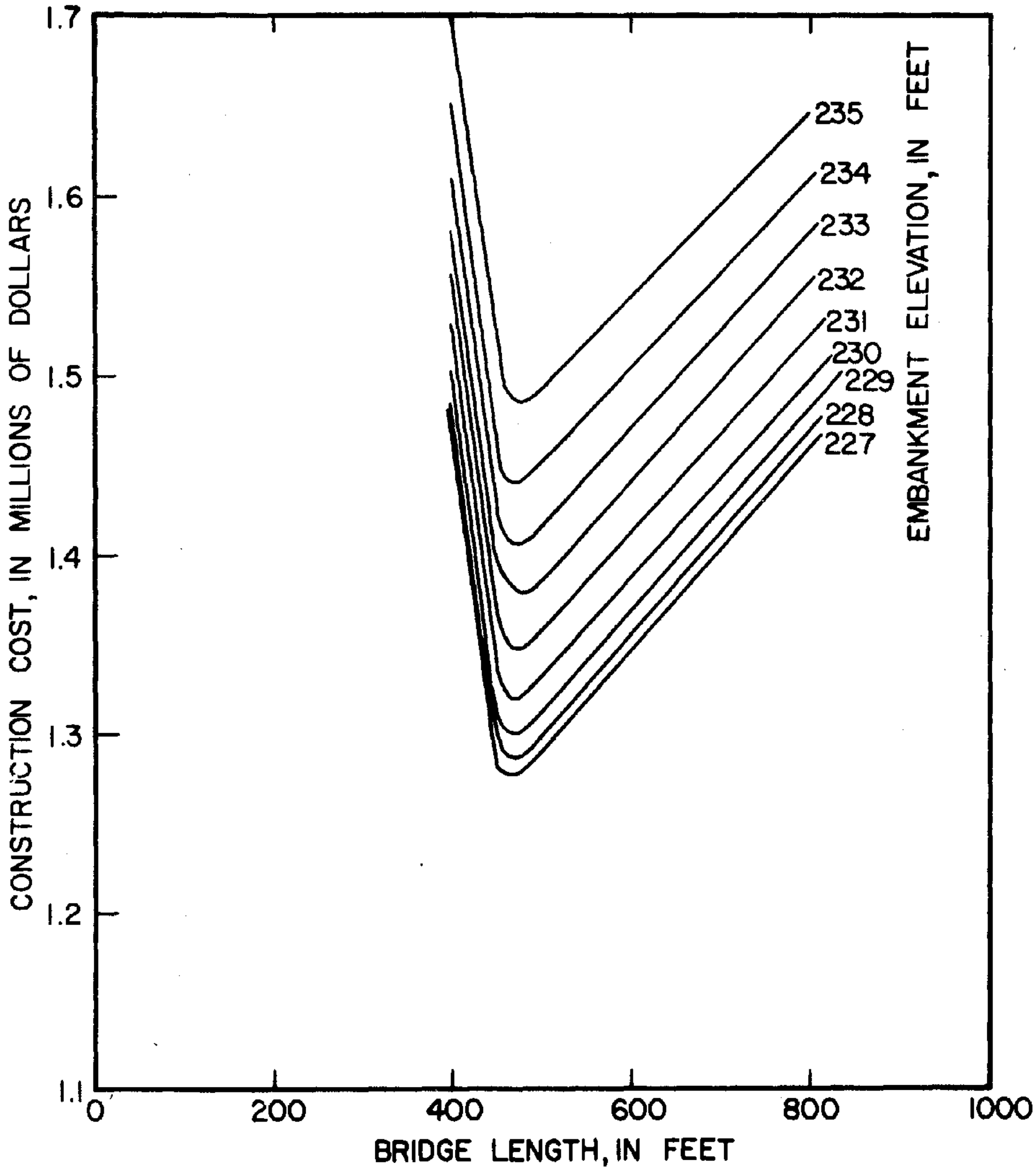

FIGURE A-14. RELATION BETWEEN CONSTRUCTION COSTS AND BRIDGE LENGTH FOR PROPOSED RURAL HIGHWAY.

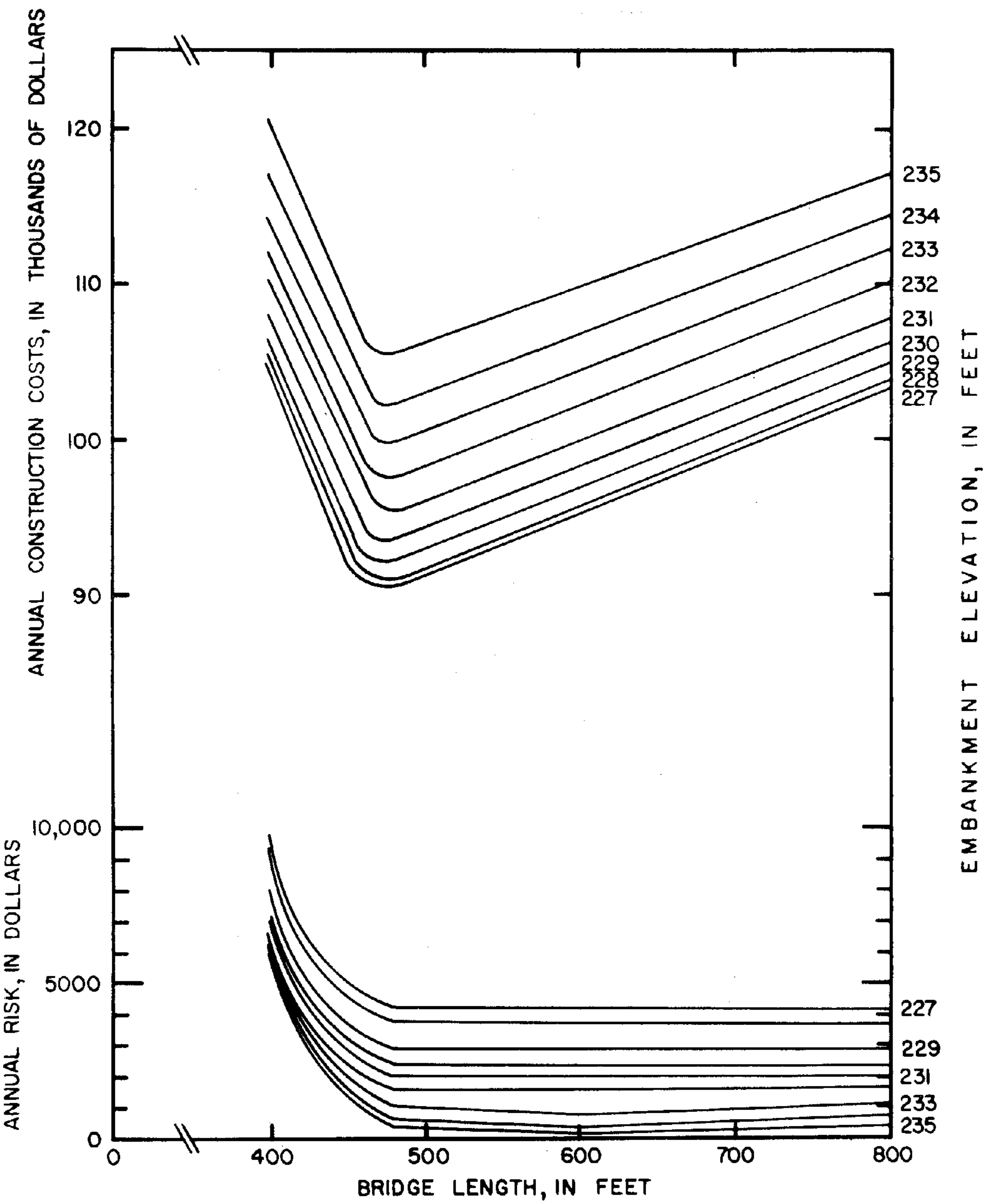

FIGURE A-15. ANNUAL CONSTRUCTION COST AND RISK AS A FUNCTION OF BRIDGE LENGTH AND EMBANKMENT.

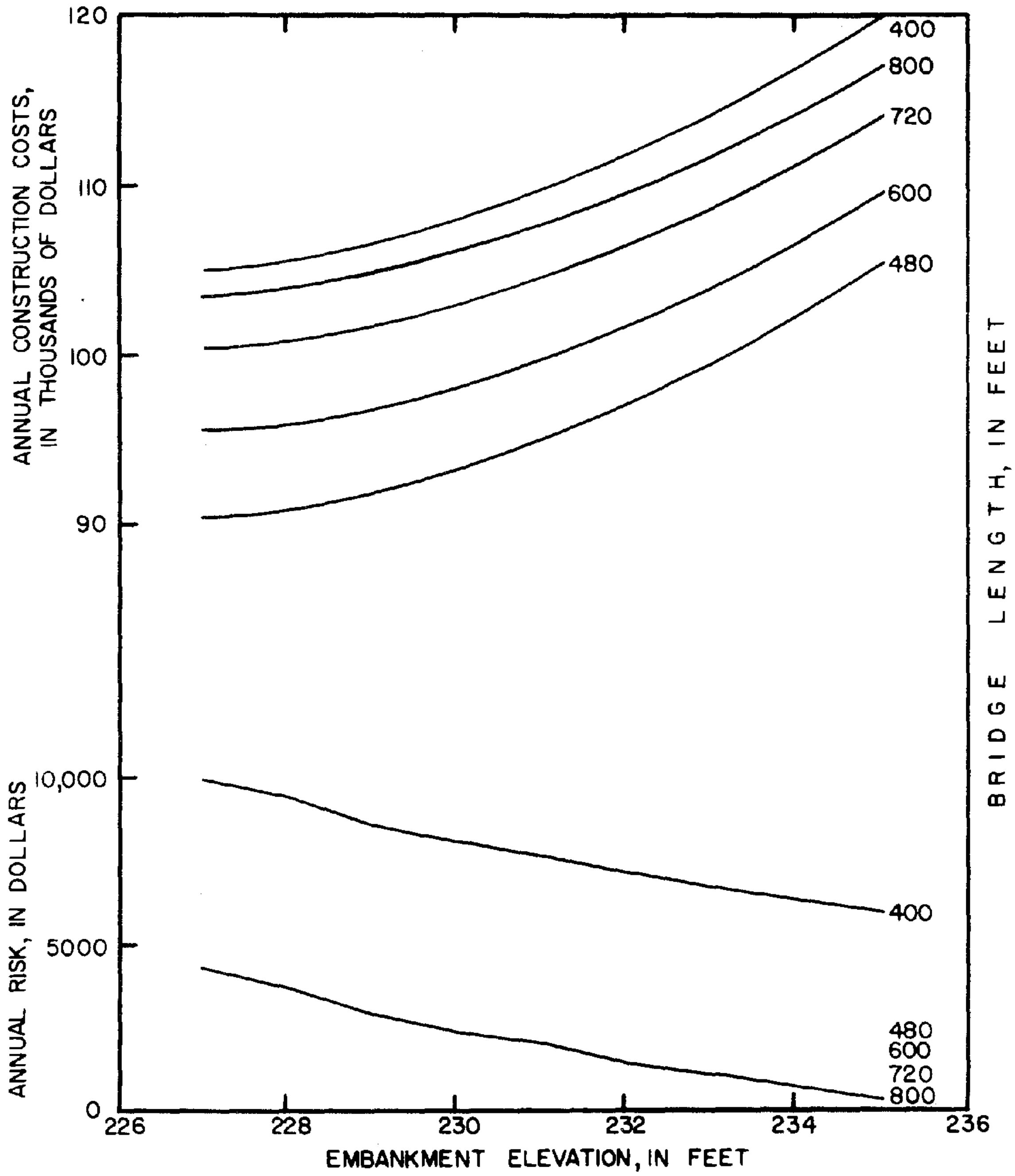

FIGURE A-16. ANNUAL CONSTRUCTION COST AND RISK AS A FUNCTION OF EMBANKMENT ELEVATION AND BRIDGE LENGTH.

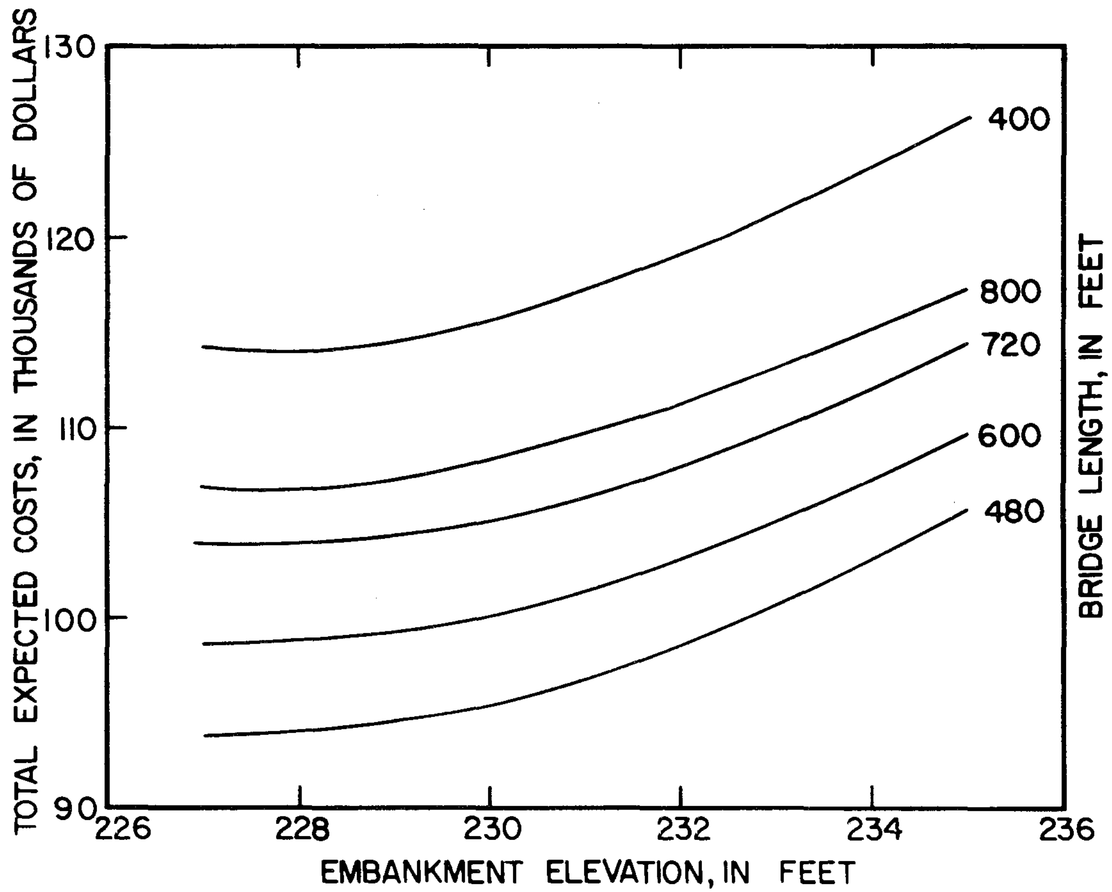

FIGURE A-17. TOTAL EXPECTED PROJECT COST AS A FUNCTION OF EMBANKMENT ELEVATION AND BRIDGE LENGTH.

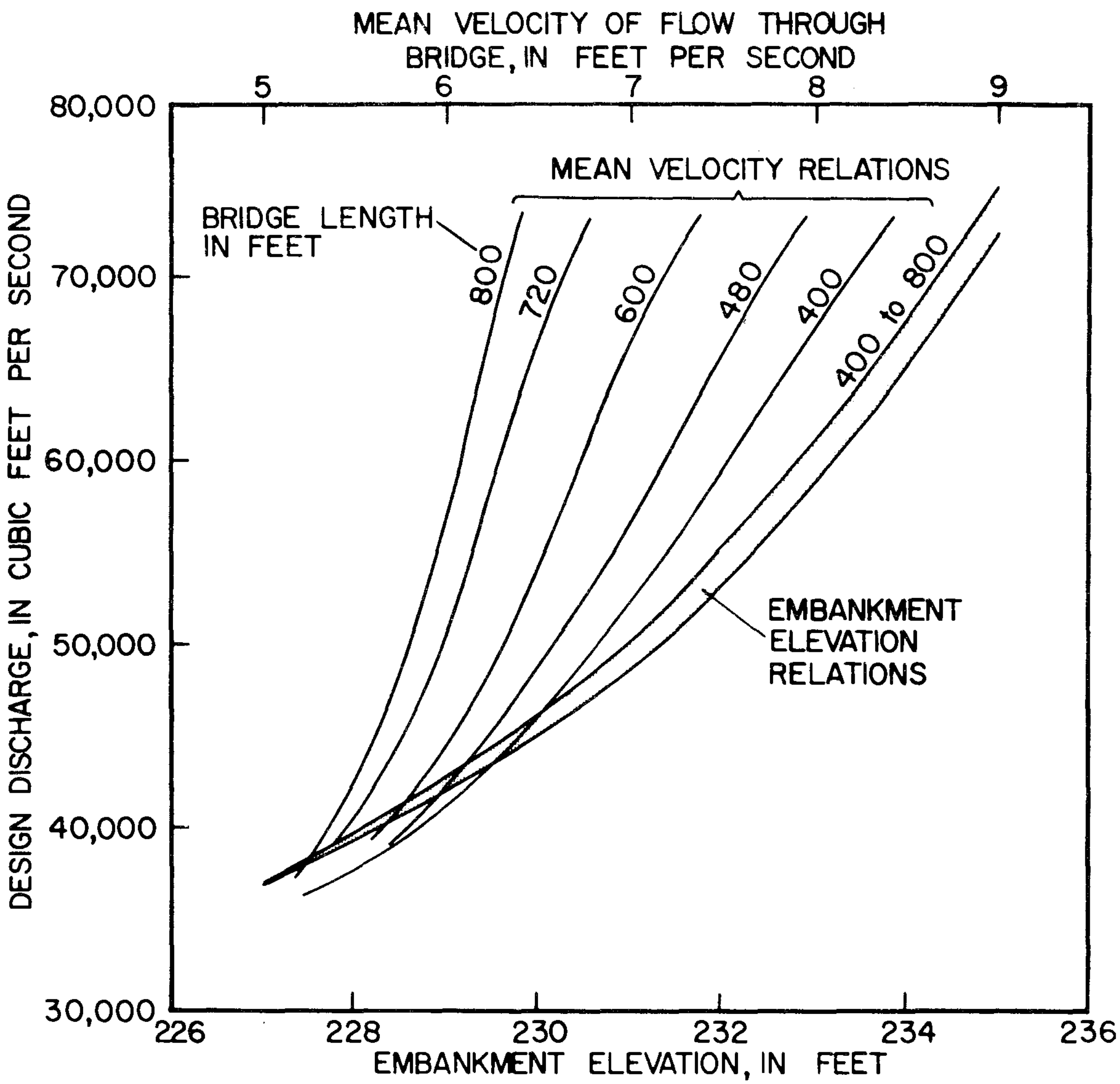

FIGURE A-18. OVERTOPPING DISCHARGE AS A FUNCTION OF EMBANKMENT ELEVATION AND BRIDGE LENGTH.

6,500 ft wide, the bridge length has only a small effect on the design discharge. However, the embankment elevation has a strong influence and the design discharge varies from 37,000 ft^3/s at an embankment elevation of 227 ft to 74,000 ft^3/s at 235 ft.

The design discharge information in figure A-18 and the discharge frequency relation in figure A-6 are utilized to compute TEPC (total expected project cost). This is plotted as a function of the probability of exceedance of the flood peak in figure A-19. The combination with the lowest TEPC is a 480-ft bridge with approach embankments at 227-ft minimum elevation. The design discharge is 37,000 ft^3/s, and the exceedance probability from figure A-6 is 0.048.

If this bridge were designed for the 2-percent-chance flood, the discharge would be 47,200 ft^3/s. This would be the design discharge for an embankment elevation of approximately 231 ft and a 480-ft bridge. The comparative costs for designing for the 4.8- and 2-percent chance floods are shown in table A-7.

The costs of channel protection for a 400-ft bridge exceed the savings from constructing the shorter span. In this case, the most economical bridge is derived by considering construction costs. Risk has little influence on the selection of the design discharge.

During a 1-percent-chance flood for the 480-ft bridge with approach embankments of 227 ft, 18,873 ft^3/s flows through the main channel bridge and 37,227 ft^3/s flows over the embankment. From figure A-18, the mean velocity for the 227-ft embankment elevation and the 480-ft bridge is 2.2 ft/s. Previous experience with the old bridge at this location suggests that this crossing design would safely pass the 1-percent-chance flood.

SUMMARY AND CONCLUSIONS

The design procedure outlined in this report which uses a risk analysis indicates that a 480-ft-long bridge with a minimum embankment elevation of 227 ft is the minimum cost bridge for the factors considered. The factors include construction cost, backwater damage, traffic interruption, and embankment repairs. The design discharge for the embankment elevation of 227 ft would be the 4.8-percent-chance (21-year) flood.

The 480-ft bridge with a minimum embankment of 231 ft which has a design discharge of the 2-percent-chance (50-year) flood costs about 3 percent more ($3,200 per year) due principally to the sparse development on the upstream flood plain.

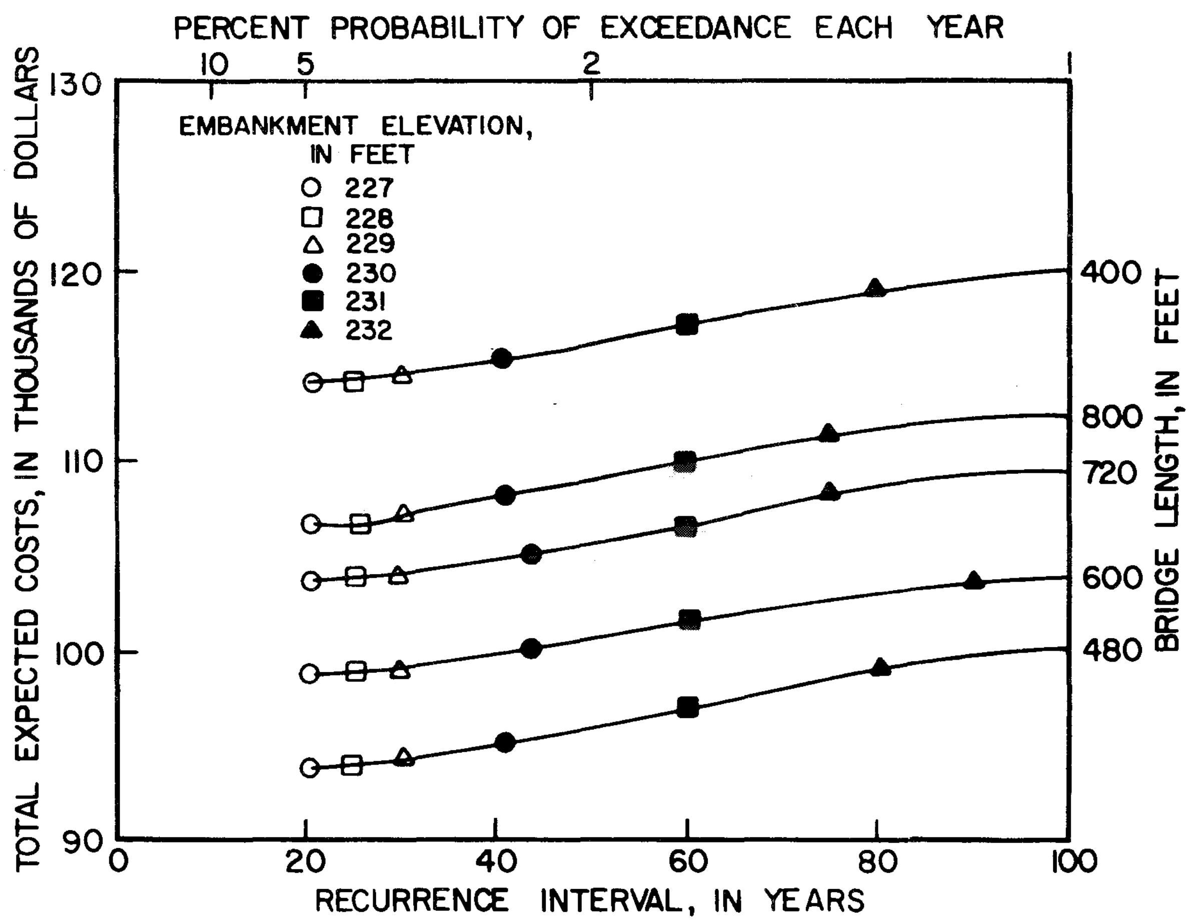

FIGURE A-19. TOTAL EXPECTED PROJECT COST AS A FUNCTION OF RECURRENCE INTERVAL AND EMBANKMENT ELEVATION.

Table A-7.--Comparison of the costs of designing for the 4.8- and 2-percent-chance floods

Flood	Bridge length (ft)	Embankment elevation (ft)	Total construction cost (dollars)	Annual cost (dollars)	Risk (dollars)	TEPC (dollars)
2 percent chance	480	231	1,342,000	95,200	1,800	97,000
4.8 percent chance	480	227	1,273,000	90,300	3,500	93,800
Difference			69,000	4,900	-1,700	3,200

The final choice of a design flood is beyond the scope of this example report. The final decision will include consideration of the information developed by a report such as this, along with budgetary constraints, the need for emergency supply and evacuation routes, the need for emergency vehicle access, and other specific site considerations.

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Appendix B

Hydraulic Design of a Bridge with Risk Analysis
At U.S. Highway 11 at Hattiesburg, Mississippi

Example Study and Report

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Hydraulic Design of a Bridge with Risk Analysis
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INTRODUCTION

Acknowledgment

This report contains the results of a sample study of the hydraulic design of a bridge using risk analysis. The particular site reported, Leaf River at U.S. Highway 11 at Hattiesburg, Mississippi, is an example of an urban site, that is, one where backwater caused by the highway embankment could cause significant damage to property located on the flood plain by increasing the backwater level. The results and methodology shown may be used to design and analyze similar sites.

The results reported are based in part on the actual data available at this crossing. Some of the site data has been modified in order to better demonstrate the influence of risk on design. Therefore, the results presented in this report should not be interpreted as being an honest evaluation of what the highway department should build at this location.

Purpose of this Study

The purpose of this study is to use risk analysis to determine the hydraulic design of a stream crossing which has the lowest TEPC (total expected project cost), that is, the optimum economic design.

Project Description

U.S. Highway 11, previously a national artery between New Orleans and Washington, D.C., became a local highway after construction of Interstate Highway 59. That segment of U.S. Highway 11 crossing Leaf River in Hattiesburg is now a city artery and has a traffic volume exceeding 12,000 vehicles per day.

At present there is a 989-ft main-channel bridge composed of a 207-ft main-channel span flanked by three 33-ft approach spans on the west end and 36 19-ft approach spans on the east end. These approach spans are on timber-pile bents and the main span is on concrete piers with spread footings set on a timber pile foundation. West of the main-channel bridge, the present highway is at flood plain level of 147 ft and east of the bridge, there is a small amount of fill that is overtopped by floods exceeding about 148.5 ft.

The existing highway alinement and the proposed realinement in the vicinity of the main channel is shown in figure B-1, the U.S. Geological Survey topographic map of the vicinity. The new alinement leaves the old alinement about 1,400 ft west of the main channel and parallels it along the upstream side, crossing the main channel about 150 ft upstream from the present bridge and rejoining the old alinement about 2,000 ft east of the main channel. It again leaves the old alinement about 2,700 ft east of the main channel, turns east and follows the existing route of State Highway 42 through Petal. The alinement is a nearly normal crossing of the flood plain and channel.

DATA COLLECTION

Flood-Plain Geometry

Seven valley cross sections of the flood plain were surveyed approximately 2,600 and 6,000 ft downstream, along the upstream and downstream sides of the proposed highway, and 5,300 and 11,000 ft upstream of the proposed highway crossing. The bridge and railroad geometry was surveyed in addition to valley cross sections upstream and downstream at the Southern Railroad 2,600 ft downstream. These data were used in the hydraulic analysis to compute the water-surface profiles. The location of the cross sections is shown on the aerial photograph in figure B-2 and the sections are shown in figures B-3A - B-3G. Figure B-3E shows a typical highway section used in this study.

Land Use

The valley is approximately half open and half wooded with cleared fields and pasture land occasionally reaching the banks of the river. In the immediate vicinity of U.S. Highway 11, the flood plain is mostly cleared upstream and downstream. There is a strip of woods 500- to 1,000-ft wide along the east bank both upstream and downstream and along the west bank downstream. The crossing is located in the developed areas of Hattiesburg and Petal, and there are numerous houses which may be affected by backwater created by the proposed crossing.

Hydraulic and Hydrologic Data

The U.S. Geological Survey has operated a continuous-record gaging station at this site since 1939 and the National Weather Service has operated a gage at this site since 1904. The largest flood occurred April 15, 1974, and crested at an elevation of 152.3 ft. The peak

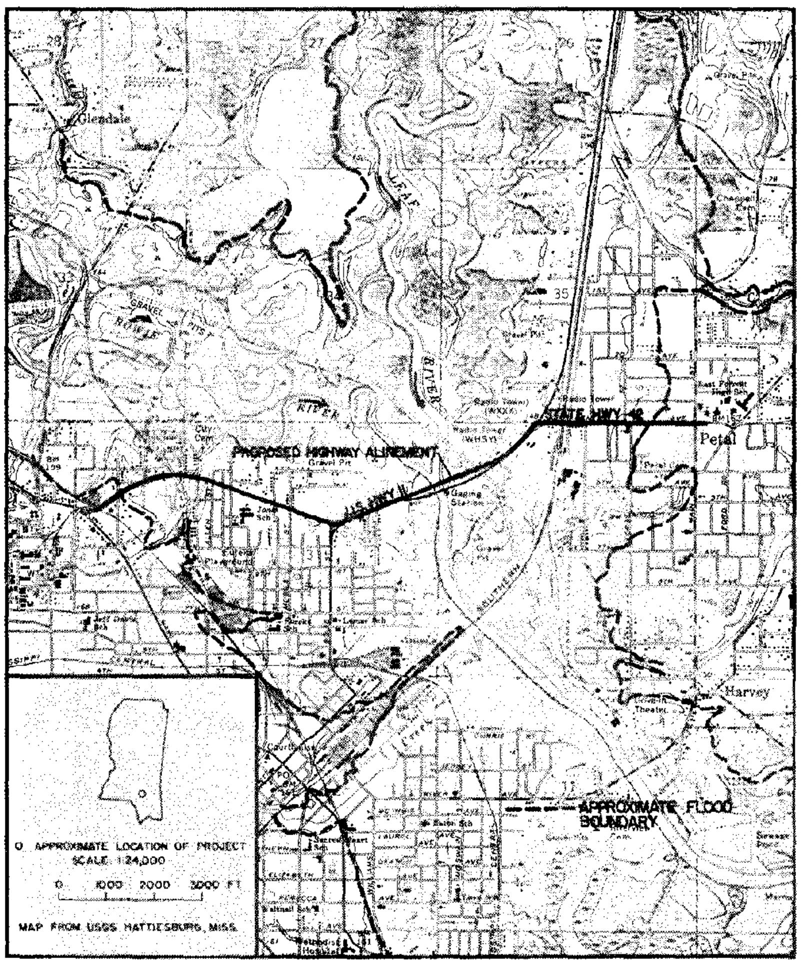

FIGURE B-1. GENERAL LOCATION MAP, LEAF RIVER AT U.S. HIGHWAY 11 AT HATTIESBURG, MISS.

FIGURE B-2. LOCATION OF THE CROSS SECTIONS USED IN THE HYDRAULIC ANALYSIS.

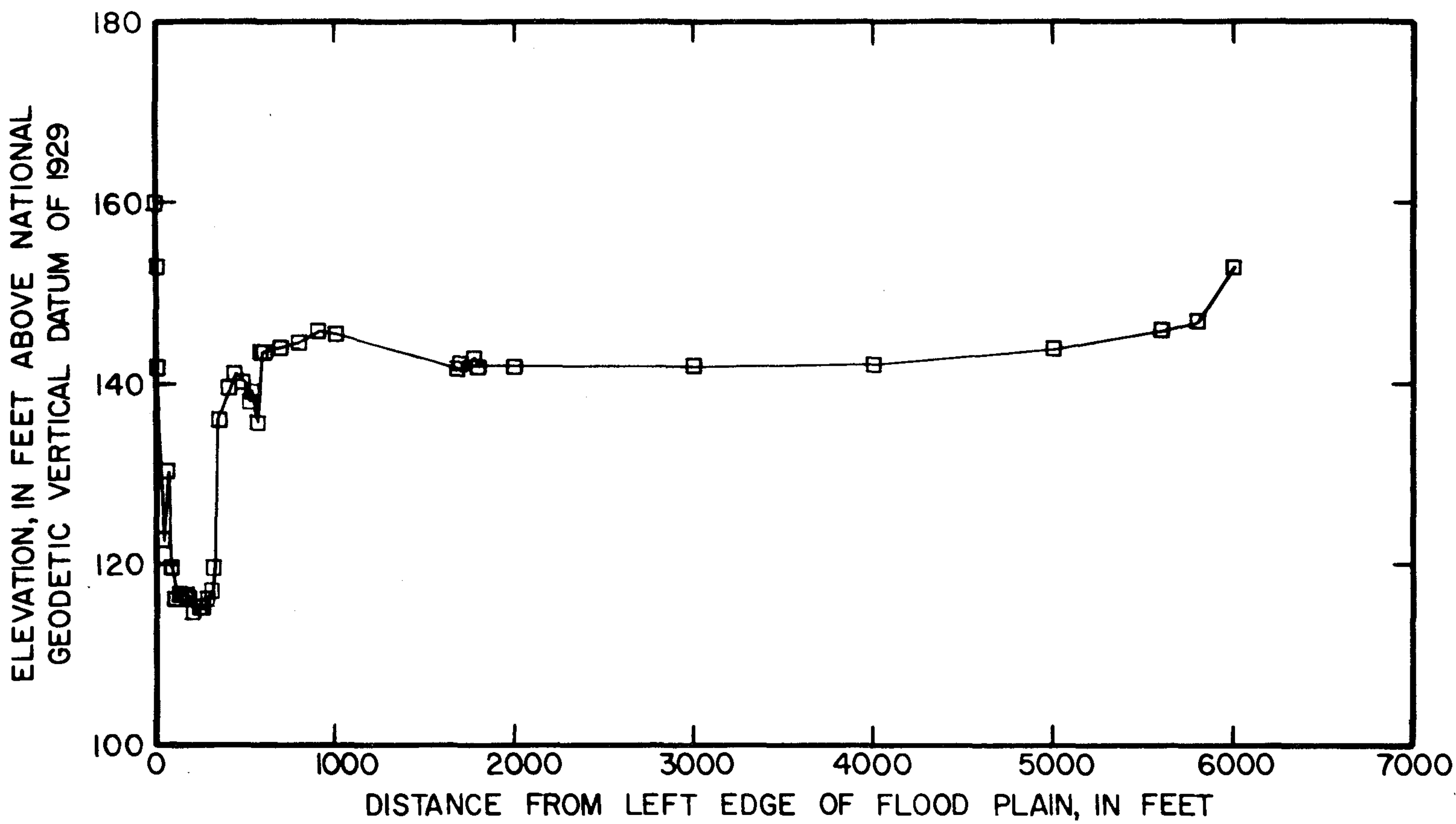

FIGURE B-3A. CROSS SECTION 1 LOCATED UPSTREAM FROM RIVER AVENUE (STATION 0).

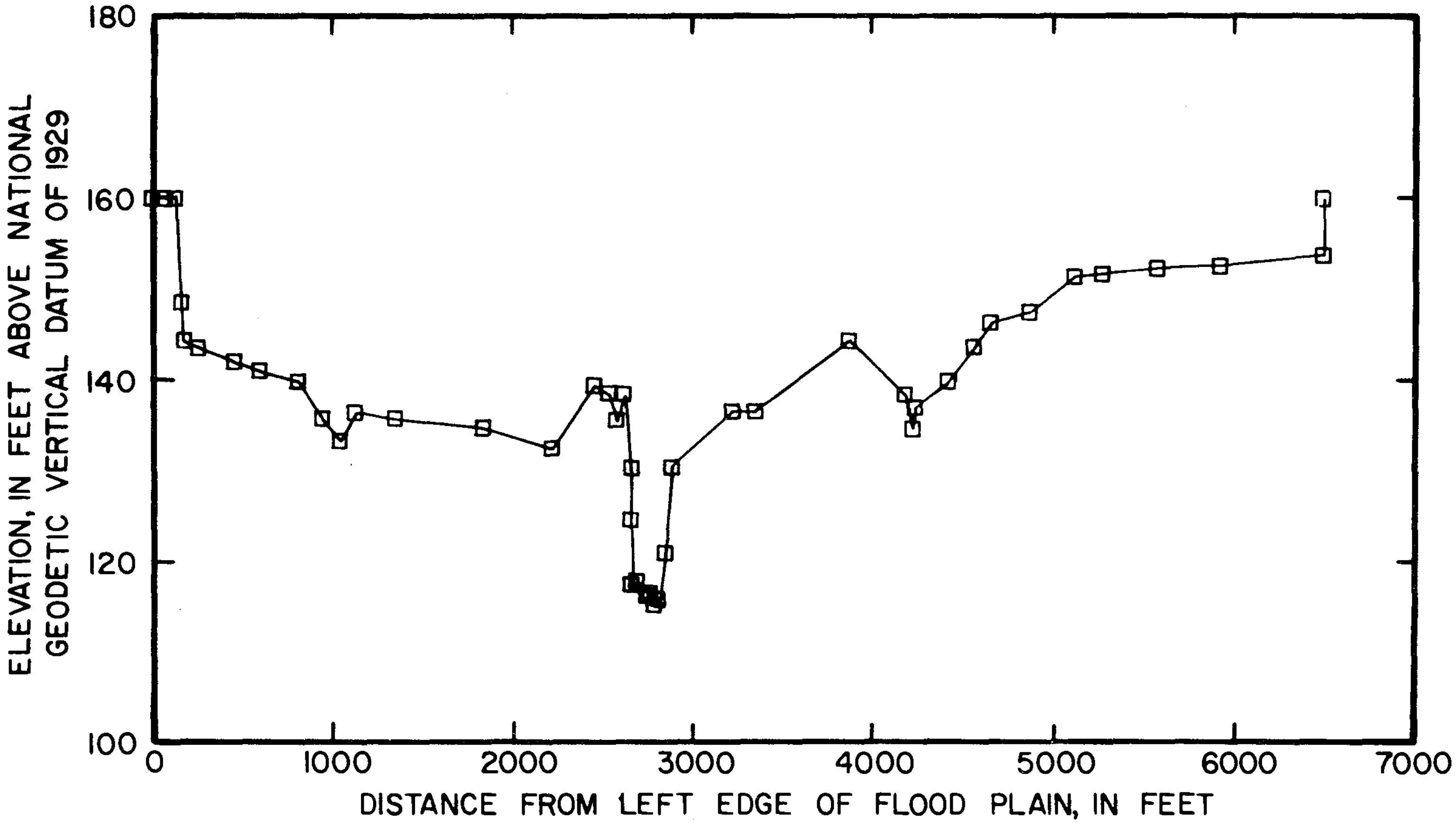

FIGURE B-3B. CROSS SECTION 2 LOCATED DOWNSTREAM FROM THE SOUTHERN RAILROAD BRIDGE (STATION 3300 FT).

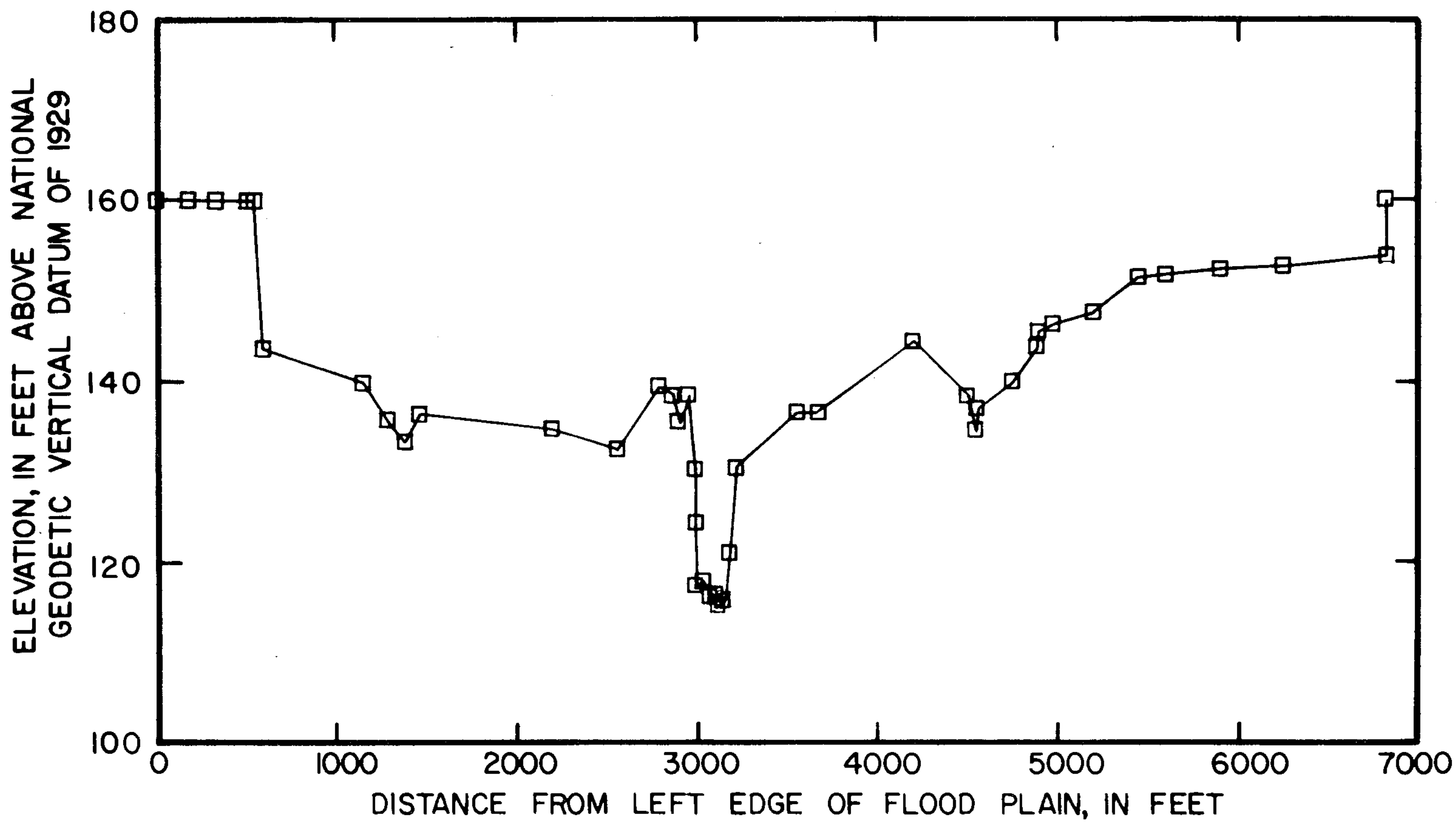

FIGURE B-3C. CROSS SECTION 3 LOCATED UPSTREAM FROM THE SOUTHERN RAILROAD BRIDGE (STATION 4500 FT).

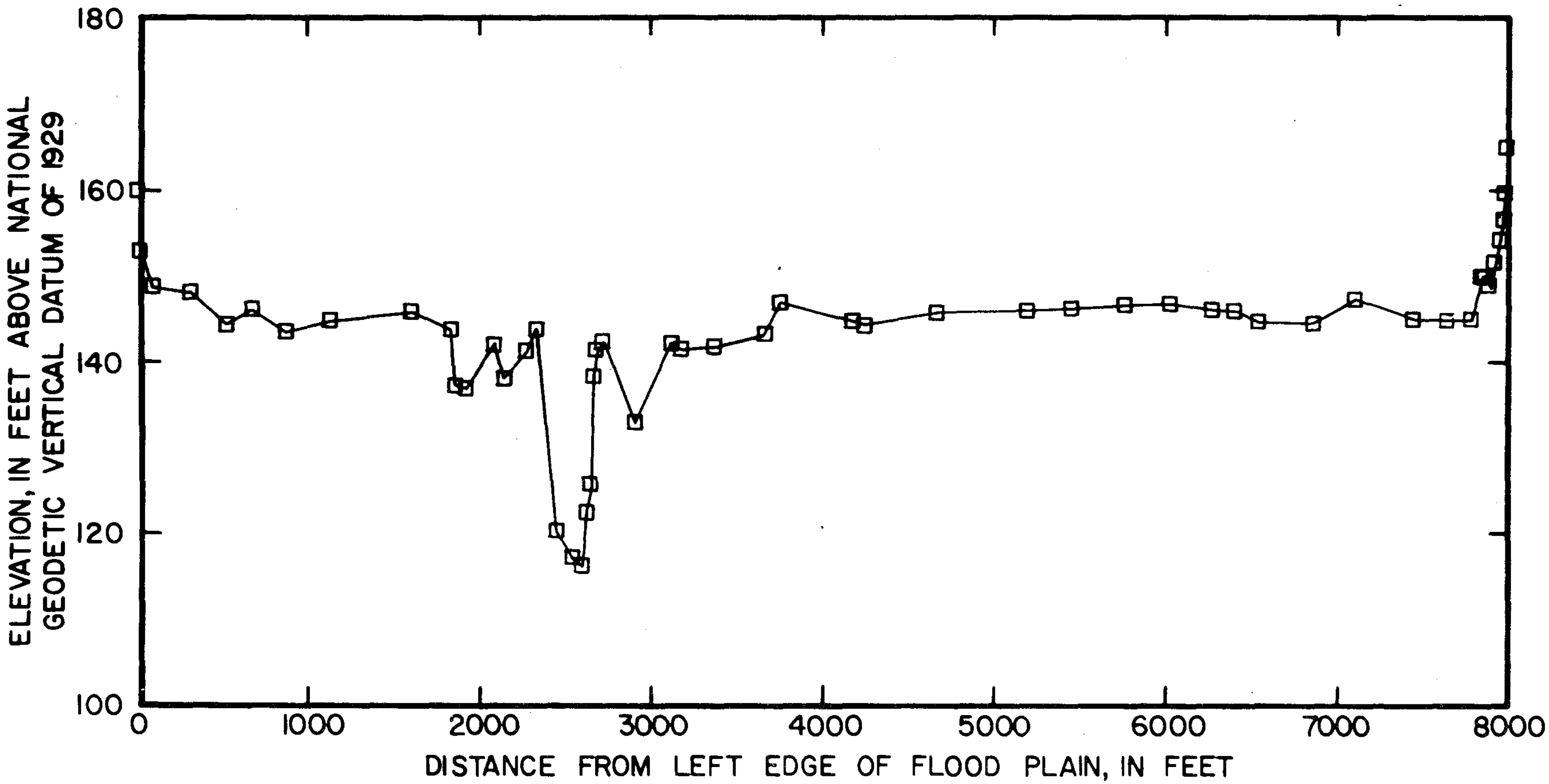

FIGURE B-3D. CROSS SECTION 4 LOCATED DOWNSTREAM FROM THE PROPOSED U.S. HIGHWAY 11 (STATION 5900 FT).

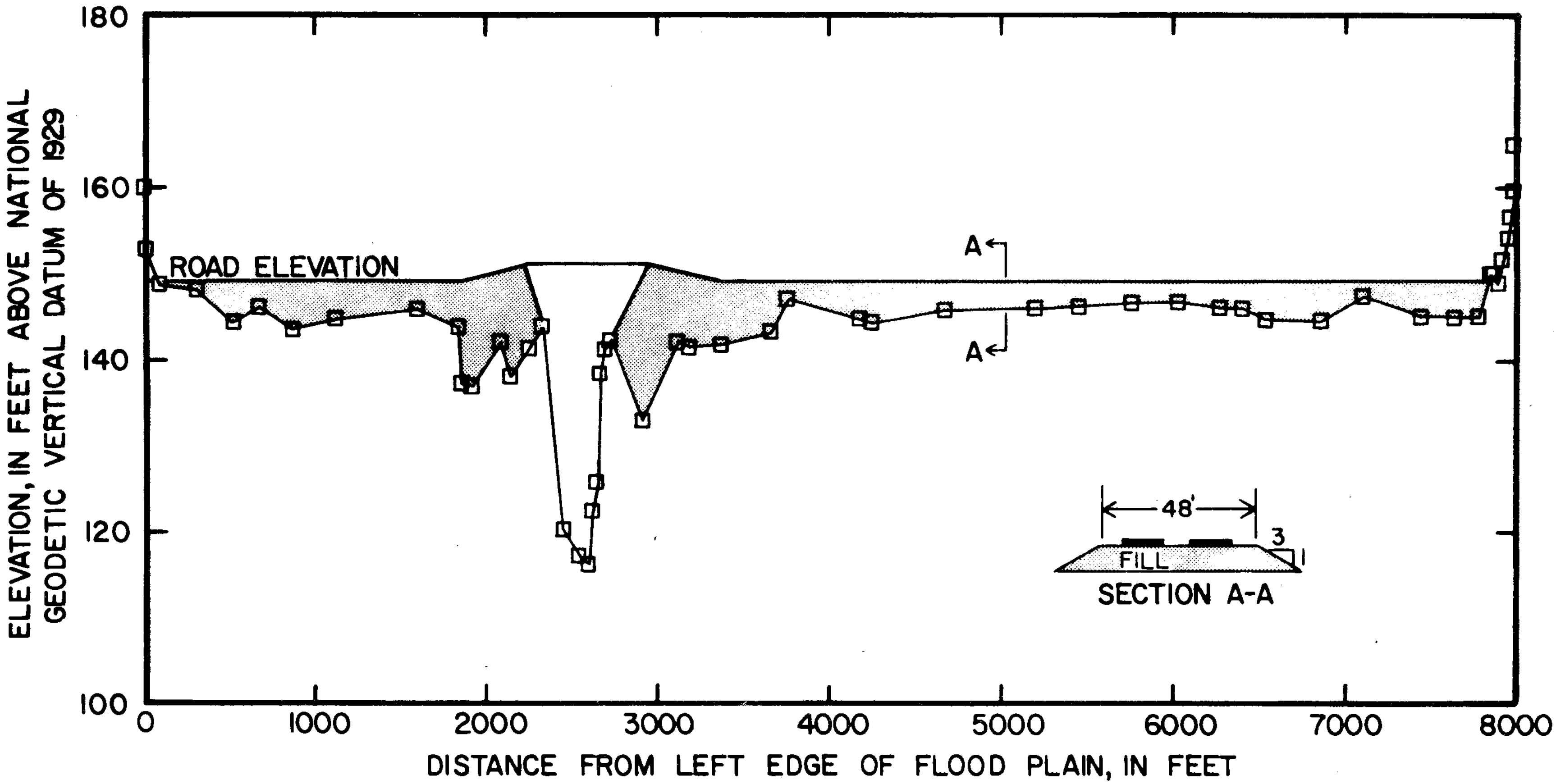

FIGURE B-3E. CROSS SECTION 5 LOCATED UPSTREAM FROM THE PROPOSED U.S. HIGHWAY 11 (STATION 6840 FT) WITH A TYPICAL HIGHWAY SECTION SUPERIMPOSED.

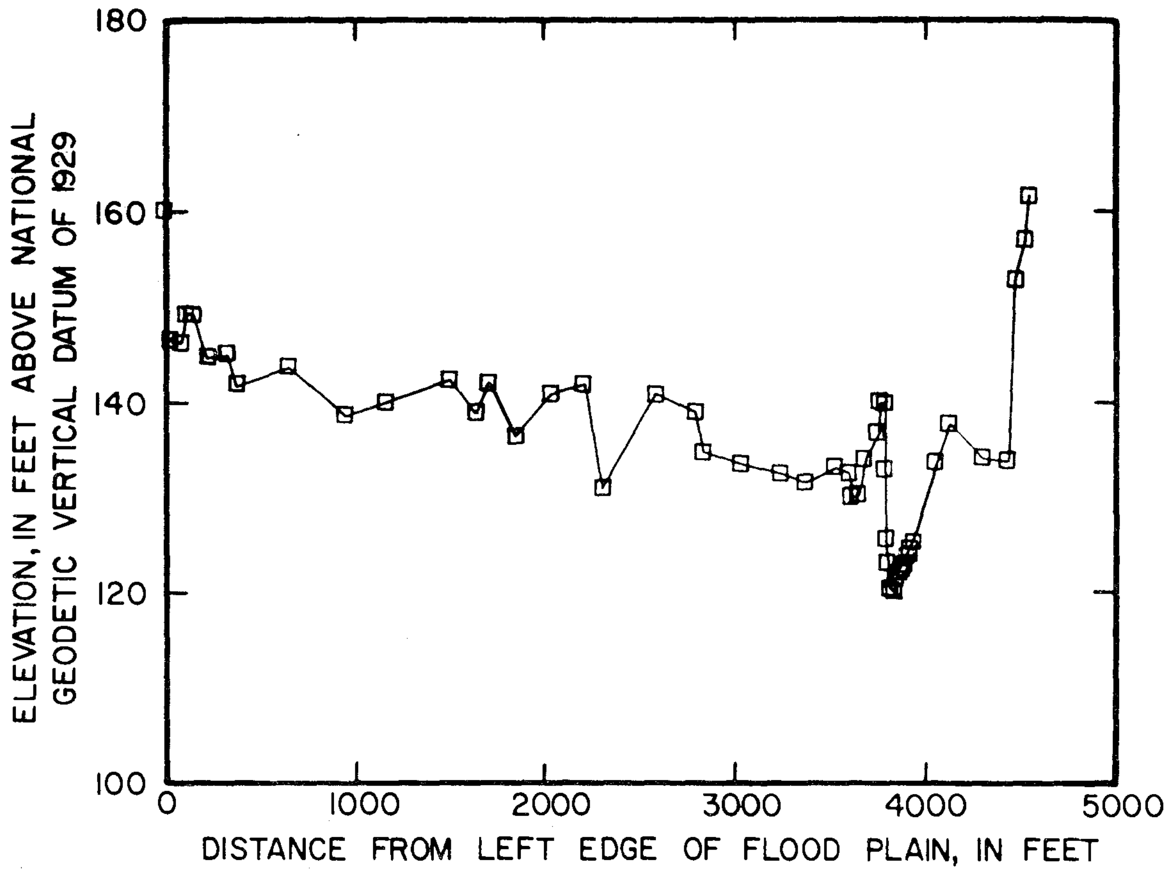

FIGURE B-3F. CROSS SECTION 6 LOCATED ON THE LEAF RIVER UPSTREAM FROM THE CONFLUENCE WITH BOWIE RIVER (STATION 11,200 FT).

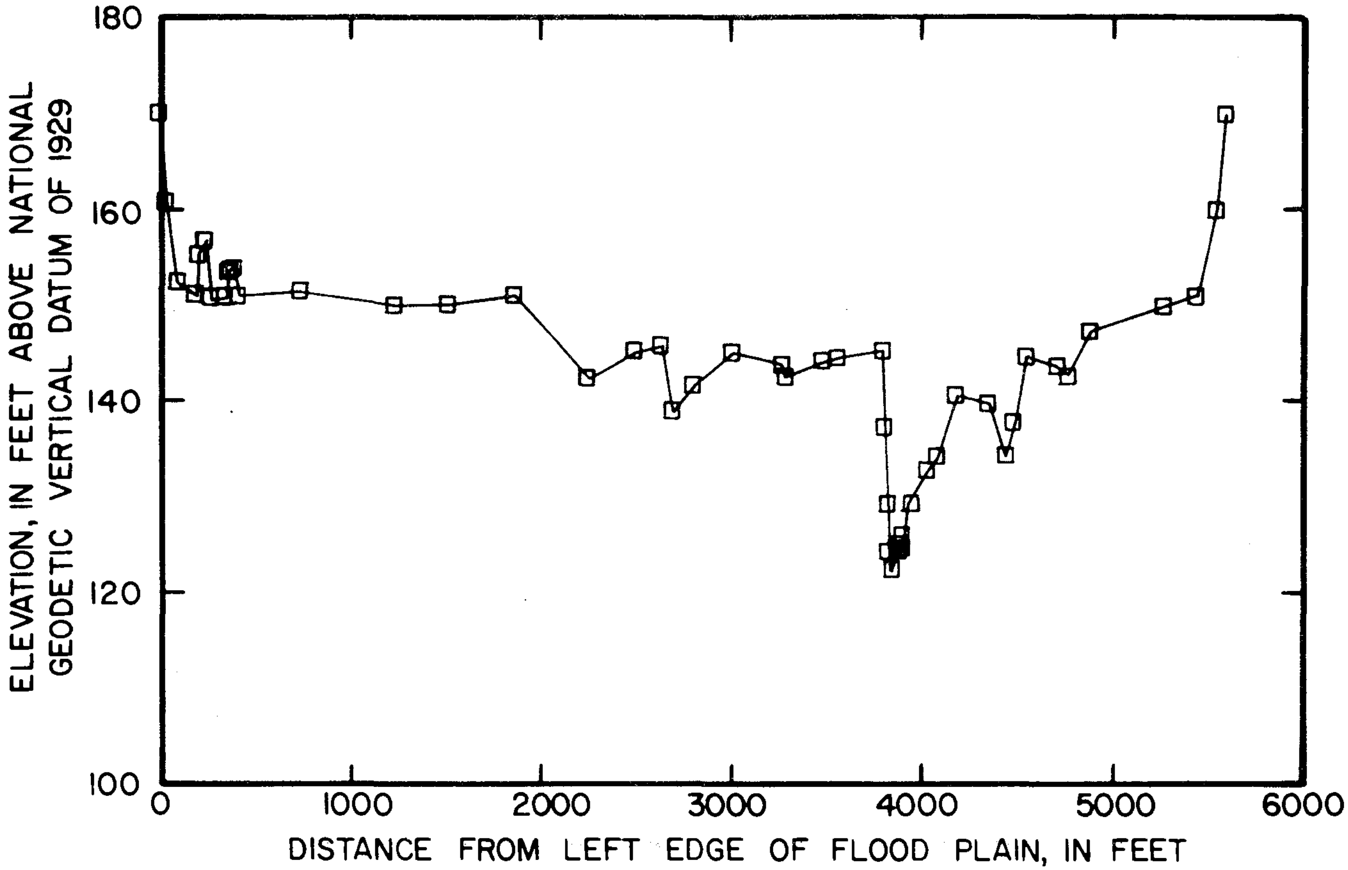

FIGURE B-3G. CROSS SECTION 7 LOCATED ON THE LEAF RIVER, 5780 FT UPSTREAM FROM CROSS SECTION 6 (STATION 16,980 FT).

discharge of this flood was 121,000 ft^3/s. The second largest known flood at this site occurred in April 1900 and crested at an elevation of 151.8 ft according to reports of the National Weather Service. Large floods also occurred in 1919, 1921, 1943, and 1961. The February 1961 flood crested at an elevation of 149.8 ft, and the peak discharge was 72,200 ft^3/s. The 1919, 1921, and 1943 peak discharges were 87,900, 82,800 ft^3/s, and 71,300 ft^3/s, respectively.

The floods of February 1961 and April 1974 overtopped the present road grade (elevation 147 ft) on the west flood plain to maximum flood depths of 3 and 6 ft, respectively. Water was over the highway for about 3 days during each of these floods.

On February 18, 1961, a maximum discharge of 72,000 ft^3/s was measured at this crossing. Of the total, 16,200 ft^3/s overflowed U.S. Highway 11 on the west flood plain, and 8,800 ft^3/s overflowed U.S. Highway 11 and State Highway 42 on the east flood plain.

During the extreme flood of April 15, 1974, a discharge of 121,000 ft^3/s was measured at the River Avenue crossing about 7,000 ft downstream. Of this total 46,600 ft^3/s overflowed River Avenue and the remainder flowed through the River Avenue Bridge. River Avenue is constructed at flood-plain level.

Geologic Data

Surface Geology

Surface geology in the vicinity is the Hattiesburg Formation which consists of perhaps 200 ft of clay, sandy clay, sand and gravel, with some thin ferruginous (containing iron) layers. The beds of sand and gravel are capable of conveying large quantities of ground water.

Channel Morphology

The 9,000-ft-wide valley is relatively flat and slopes gently upward near the edges of the flood plain. The large well-defined channel conveys the majority of the discharge of large floods. The 400-ft-wide channel has only very minor meanders in the 1.5-mile-reach upstream from the proposed crossing but meanders moderately above and below this reach.

Leaf River at U.S. Highway 11 has been classified by Brice (1975) as "Stream Class Symbol 2C". This indicates a degree of sinuosity between 1.06 and 1.5 and a character of sinuosity of "single phase, wider at bends, chutes rare".

Bowie River with a drainage area of about 660 mi^2 flows into Leaf River about 400 ft upstream from the proposed highway crossing. The total drainage area of Leaf River at the crossing is 1,760 mi^2 and the valley slope is 2.5 ft/mi in the vicinity. The slope of Bowie River is about 3 ft/mi.

Soils Information

Considerable soils information has been collected at this crossing by the Mississippi Highway Department during 1975. The streambed is composed of dense sand and gravel, and the banks are formed of medium dense sand with a silty clay overburden 5 to 10 ft thick. A layer of hard blue silty clay outcrops in the streambed a few hundred feet downstream.

Scour History

The channel position has been generally stable since gaging records began in 1938. Scour and fill of as much as 10 to 12 ft has occurred in the channel and especially along the west edge of the channel. The channel appears to have been more stable during the past 10 or 15 years.

A large scour hole usually exists at the confluence of Leaf and Bowie Rivers 400 ft upstream from the site. Turbulence created by the mixing of the water from the two streams creates a potential scour problem just upstream from U.S. Highway 11. The surveyed centerline of the proposed crossing about 150 ft upstream from the existing crossing showed a bottom elevation of 104 ft compared to 116 ft at present U.S. Highway 11. These data indicate that the proposed crossing may be in the edge of a scour hole created by the confluence of the two streams. The turbulence which develops this hole may be worsened by the construction of bridge piers causing the hole to enlarge and engulf the bridge site.

Gravel and sand mining operations exist at several sites in the Leaf and Bowie River flood plains both upstream and downstream from the crossing. Some are large industries which have operated for perhaps 40 or 50 years. This activity has probably affected scour and fill at the bridge site over the years. Periodic channel flow-line surveys of both Leaf and Bowie Rivers (for perhaps a mile upstream and downstream) should be obtained for possible analysis of the effect of the mining on scour at the highway crossing.

Cost Data

Construction Costs

The Highway Department estimated construction costs for bridge lengths of 280, 440, 600, 800, 1,200, and 1,600 ft for embankment heights varying from 147 to 155 ft in 2-ft increments. Table B-1 summarizes these costs.

Costs of some bridges were increased to include an extra $500,000 for extensive protection works at both abutments where average velocities were greater than 6 ft/s. The bridges for which this extra cost was included are shown in table B-1.

Costs of elliptical spur dikes (either 100 or 150 ft long) were included for all bridges and embankment elevations of 149 to 155 ft. Costs of two 150-ft spur dikes were included for all bridges with embankment heights of 151 ft or greater.

Backwater Damage

There are 301 residences which could be damaged due to backwater from the bridges. Velocities on the flood plain in the vicinity of the residences are low (less than 1 ft/s); therefore, damages from velocity were not considered. Data were collected on types of residences, first floor elevation, location in the flood plain and value of the residences. These residences were of similar size and construction so that an average value of $75,000 was used for each unit (1977). Contents of each residence was assumed to be 50 percent of the value of the structure. Therefore, each residence had a total value of $37,500. These data along with the relation between flood depth and damages, shown in table B-2, were used to compute damages to the structures and contents caused by backwater flooding. Backwater flood damage is the difference between the damage with the highway crossing in place and the natural condition (no highway).

Traffic Data

The present ADT (average daily traffic) was determined to be 12,680 vehicles. Traffic at this site was projected to grow at the compound rate of about 2 percent per year. The equivalent uniform annual series from equation 2 for the average daily traffic is 15,774 vehicles per day. A suggested route for travel from Hattiesburg to Petal when U.S. 11 is out of service for any reason is as follows: (reverse the order for travel from Petal to Hattiesburg.)

1. Turn left on U.S. 49 at the intersection with U.S. 11.
2. Take I-59 North at its intersection with U.S. 49.
3. Exit I-59 at the Moselle exit proceeding to U.S. 11.
4. Turn right at U.S. 11 and proceed to Petal.

Table B-1.--Construction costs in thousands of dollars for various bridge lengths and embankment heights, Leaf River at U.S. Highway 11, near Hattiesburg, Mississippi

Embankment elevation (ft)	Bridge length (ft)	Bridge cost	Embankment cost	Abutment protection	Total cost
147	280	2512	126	500	3138
	440	2438	126	500	3064
	600	2779	126	500	3405
	800	3070	126	-	3196
	1200	3582	126	-	3708
	1600	4094	126	-	4220
149	280	2526	217	500	3243
	440	2452	216	500	3168
	600	2793	214	500	3507
	800	3083	211	500	3794
	1200	3774	205	-	3979
	1600	4104	199	-	4303
151	280	2527	365	500	3392
	440	2452	363	500	3315
	600	2793	357	500	3650
	800	3084	350	500	3934
	1200	3595	336	-	3931
	1600	4107	322	-	4429
153	280	2526	531	500	3563
	440	2451	534	500	3485
	600	2793	524	500	3817
	800	3083	513	500	4096
	1200	3596	489	500	4585
	1600	4107	466	-	4573
155	280	2526	734	500	3760
	440	2451	729	500	3680
	600	2793	715	500	4008
	800	3074	699	500	4273
	1200	3596	665	500	4761
	1600	4107	632	-	4739

Table B-2.--Relation between flood depth and damage due to flooding as a percentage of structural value, Leaf River at U.S. Highway 11, near Hattiesburg, Mississippi

Depth of flood above floor (ft)	Damage of structure value (percent)
0	5
1	13
2	18
3	22
4	25
5	28
6	31
7	34
8	37
9	41
10	45
11	48

The total length of the detour is 25 mi or 23 mi longer than the normal route. Analysis by the Highway Department indicates that the average additional mileage cost per trip utilizing the detour is $4.66. The cost per trip of lost time was estimated to be $1.50. The increased chance of accidents was negligible (less than $50 per day) according to Highway Department data. Use of the above figures indicate that the total additional cost of moving 15,774 vehicles on the detour is $97,168 per day.

Embankment Repairs

Highway embankments that are overtopped for sustained periods of time could be eroded. This erosion will result in traffic interruption and in the need for repair to the embankment. Some experience is available in the State Highway Department concerning the duration of overtopping needed to wash out the road to the extent that it is impassable and the length of time and cost needed to repair the damage. Figure B-4 defines the relation used to determine when a traffic stopping washout will occur.

Damage is related to velocity and fill material. Velocities greater than 1 ft/s may begin to scour the sod shoulders of a well-sodded clay fill within 1 hour, and velocities of 0.5 ft/s may begin to scour a poorly sodded sandy fill within 1 hour. Damage usually begins along the downstream shoulder but when a fill is superelevated with the upstream shoulder higher, damage may begin along the upstream shoulder.

Observations at a dozen or more sites which were overtopped 0.25 ft or less for periods of up to several days, show no appreciable damage to well-sodded fills even when composed principally of sand. When flow approaches 0.5 ft deep for a period of only 1 hour, shoulder erosion is probable and flow of 0.5 ft depth for 18 hours may result in shoulder erosion averaging 4 ft wide by 1 ft deep plus loss of asphalt pavement averaging 1 ft wide along the entire length of the fill. Observations at one site which was overtopped 0.5 ft for 2 days showed shoulder erosion averaging 10 ft wide by 4 ft deep plus loss of asphalt pavement 6 ft wide.

Flow 2 ft deep for 1 hour may result in shoulder erosion 2 ft wide and 0.5 ft deep. Flow 5 ft deep for 1 hour may result in shoulder erosion 5 ft wide and 2 ft deep. Flow 3 ft deep for 2.5 days may erode the entire embankment. Also, flow 4 ft deep for 2 days and flow 5 ft deep for 1.5 days may erode the entire embankment.

Minor repairs would probably be made by Highway Department maintenance personnel using highway equipment to haul fill material procured nearby

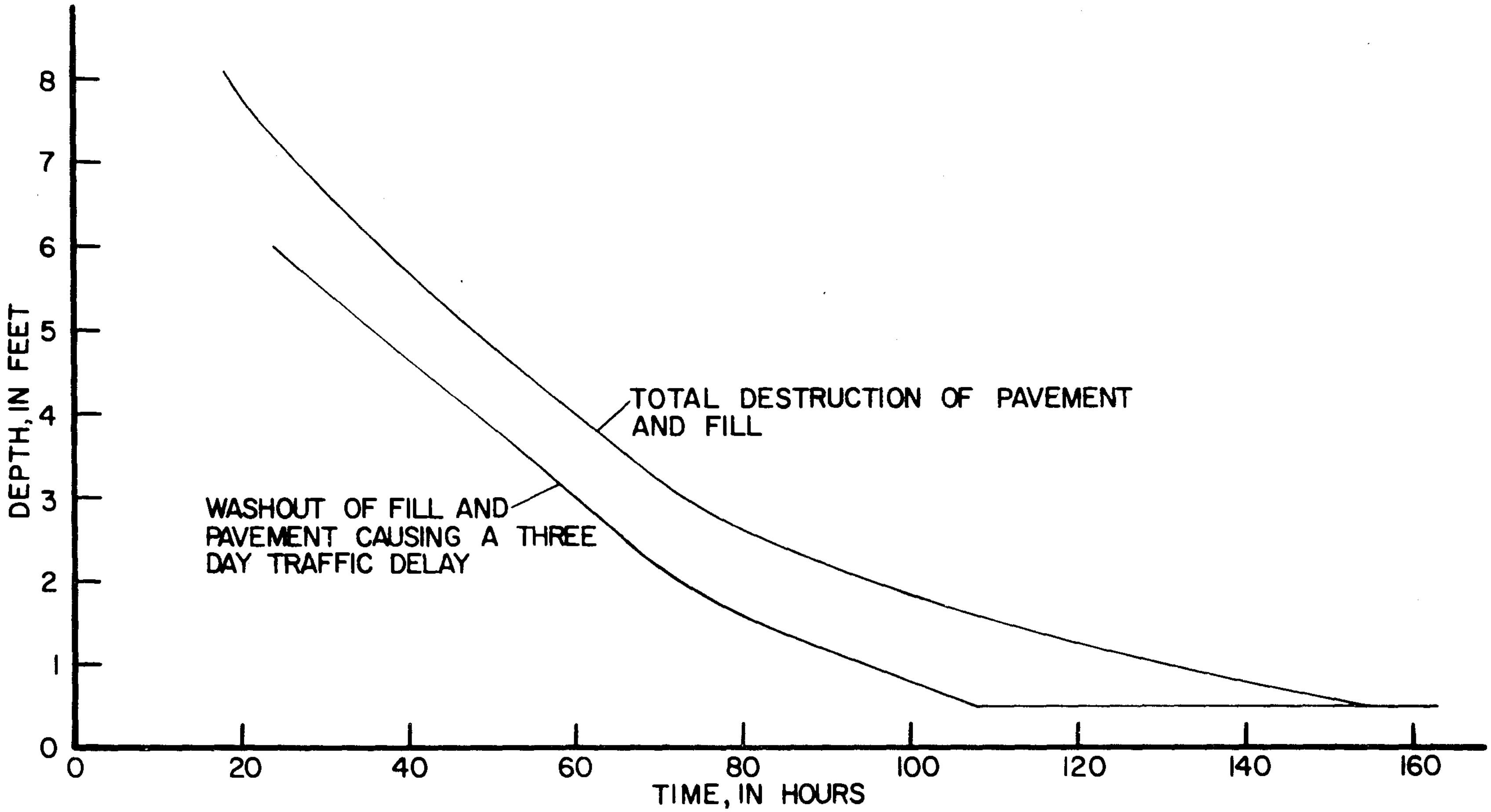

FIGURE B-4. ESTIMATED RELATION BETWEEN TIME AND DEPTH OF FLOW NECESSARY TO WASH OUT FILL AND PAVEMENT, LEAF RIVER AT U.S. HIGHWAY 11 AT HATTIESBURG, MISS.

and paving it with asphalt from a nearby mix plant. Total washouts of several hundred feet of highway would probably be repaired by an emergency contract.

Figure B-5 shows the relation between repair cost and the time and depth of overtopping for a 4-ft-high (average) embankment. It is estimated that a 3-day traffic delay will occur if the embankment is breached to the extent that traffic cannot pass. These estimates are for 100-ft length of highway with a 4-ft embankment. Adjustment coefficients (ratios of embankment costs) were used to adjust these curves for other embankment heights. This was accomplished by multiplying figures derived from figure B-4 by the ratio of the cost of the embankment being considered and the cost of a 4-ft embankment. These coefficients are shown in table B-3.

The relations shown in figure B-5 are based on the observations of a few situations described previously. The curve for an overtopping depth of 0.5 ft is defined by several observations; other curves are defined by one or two points. In constructing the curves, the following unit costs were used to convert from quantities to dollars: fill, \$2 per cubic yard and asphalt pavement, \$14.40 per square yard.

DATA ANALYSIS

Magnitude and Frequency of Floods

A log-Pearson Type 3 discharge-frequency curve (fig. B-6) for this site, which is downstream of the confluence of the Leaf River and the Bowie River, is based on the annual peak discharges for the period 1905-76 and the historical peak discharge of April 1900. The drainage area at this site is 1,760 mi^2. This frequency curve was developed using procedures outlined in the United States Water Resources Council (1977). According to figure B-6, the 2-percent-chance flood at this site is 90,500 ft^3/s, and the 1-percent-chance flood is 110,000 ft^3/s. Floods of 1961 (72,000 ft^3/s) and 1974 (121,000 ft^3/s) had exceedance probabilities of 4.2 and 0.6 percent, respectively.

A log-Pearson Type 3 discharge-frequency curve (fig. B-7) for Leaf River immediately upstream from Bowie River, where the drainage area is reduced from 1,760 mi^2 to 1,100 mi^2, is based on data for the gaging station on Leaf River near Collins (drainage area, 752 mi^2) adjusted to the 1,100 mi^2 drainage area. According to figure B-7, the 2-percent-chance flood for Leaf River above Bowie River is 59,000 ft^3/s and the 1-percent-chance flood is 70,000 ft^3/s. The peak discharges for the 1961 and 1974 floods which were estimated to have been 48,000 and 70,000 ft^3/s, respectively, have exceedance probabilities of 4.2 and 0.6 percent, respectively (fig. B-7).

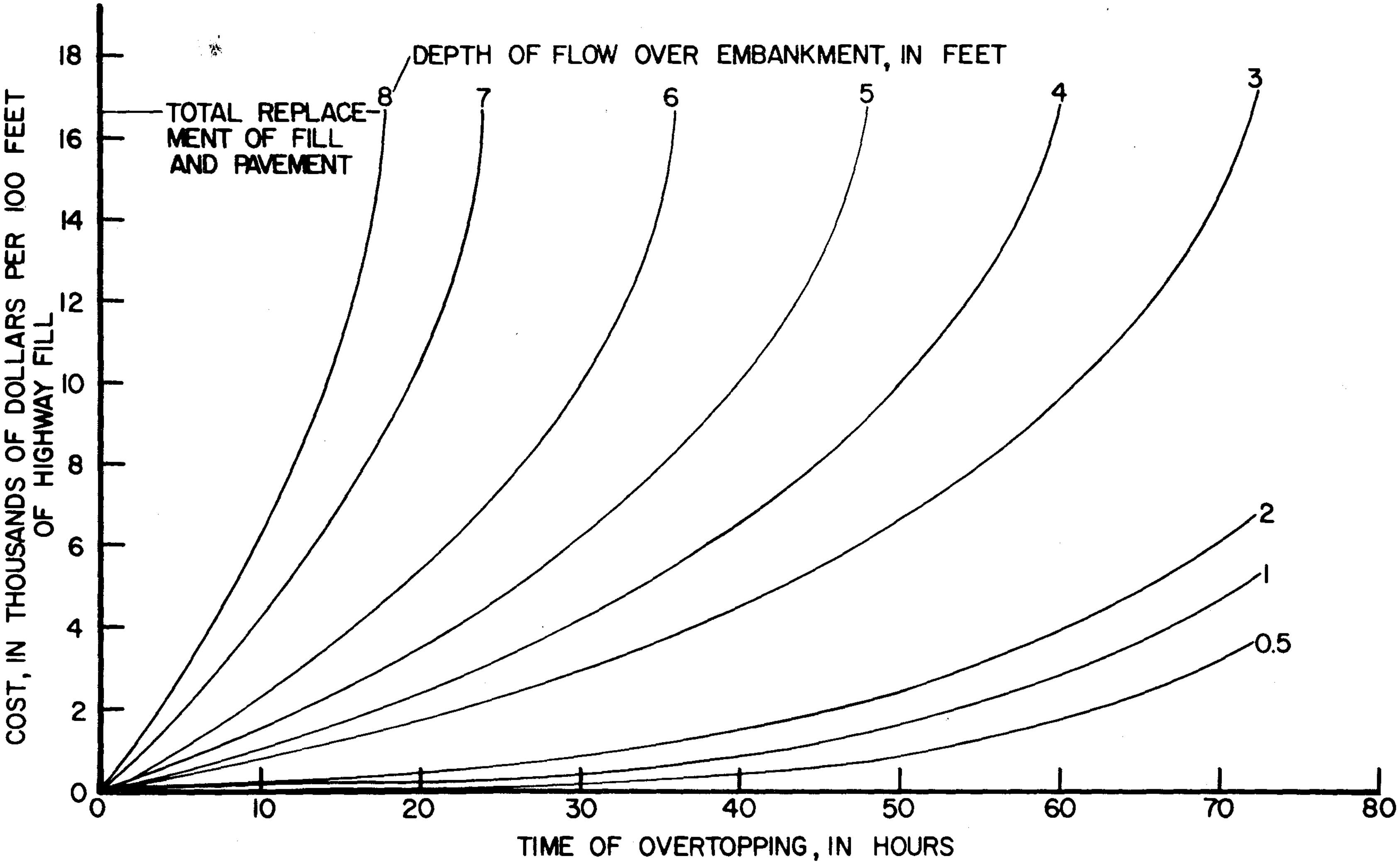

FIGURE B-5. COST OF REPAIRING A 48-FOOT ROAD AND FILL 4 FEET HIGH.

Table B-3.--Ratios of embankment costs to 4-ft embankment costs, Leaf River at U.S. Highway 11 at Hattiesburg, Mississippi

Embankment elevation (ft)	Adjustment coefficient
147	0.37
149	0.61
151	1.00
153	1.46
155	1.99

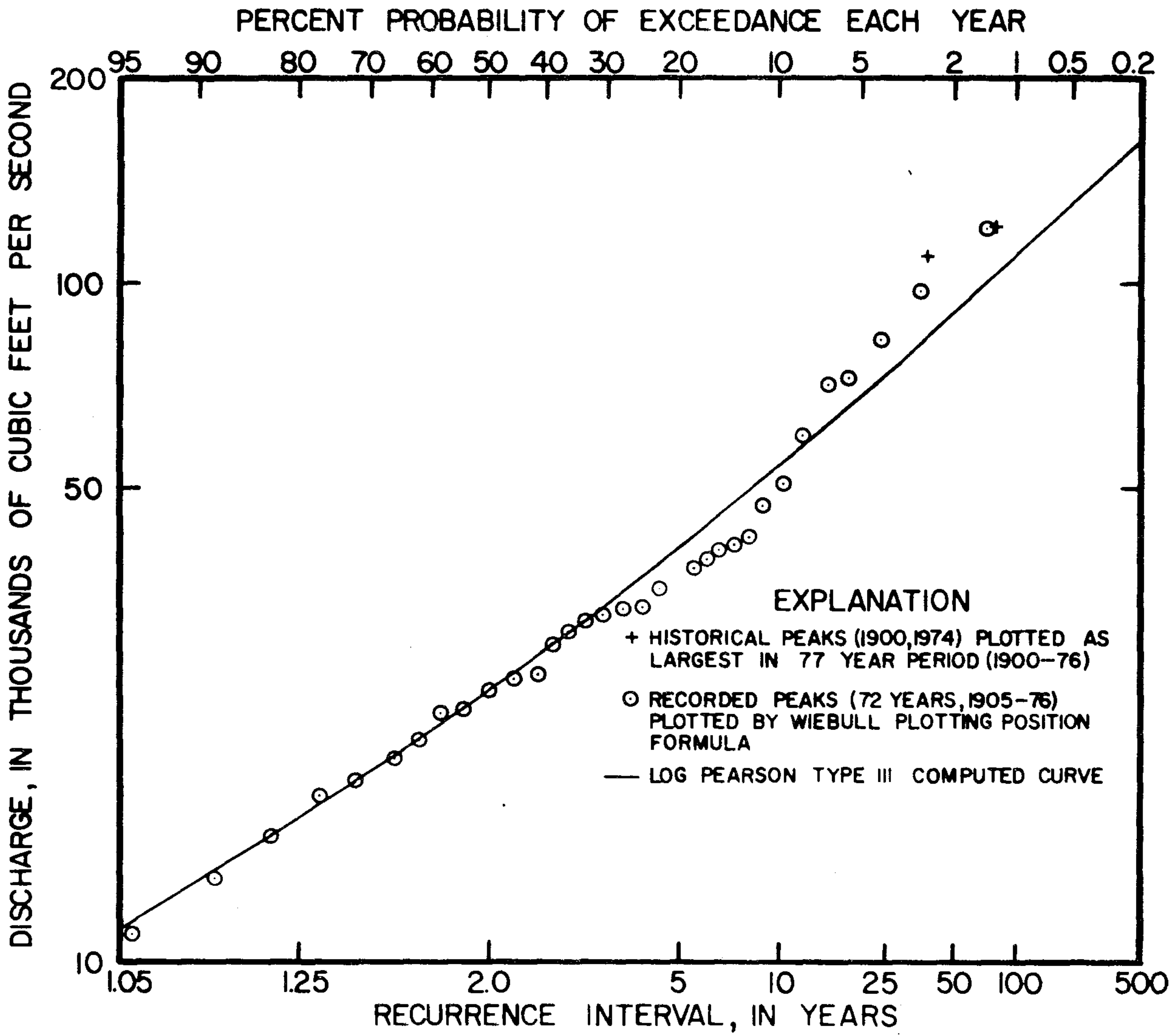

FIGURE B-6. MAGNITUDE AND FREQUENCY OF FLOODS, LEAF RIVER AT U.S. HIGHWAY 11 AT HATTIESBURG, MISS. (DRAINAGE AREA 1760 SQUARE MILES.)

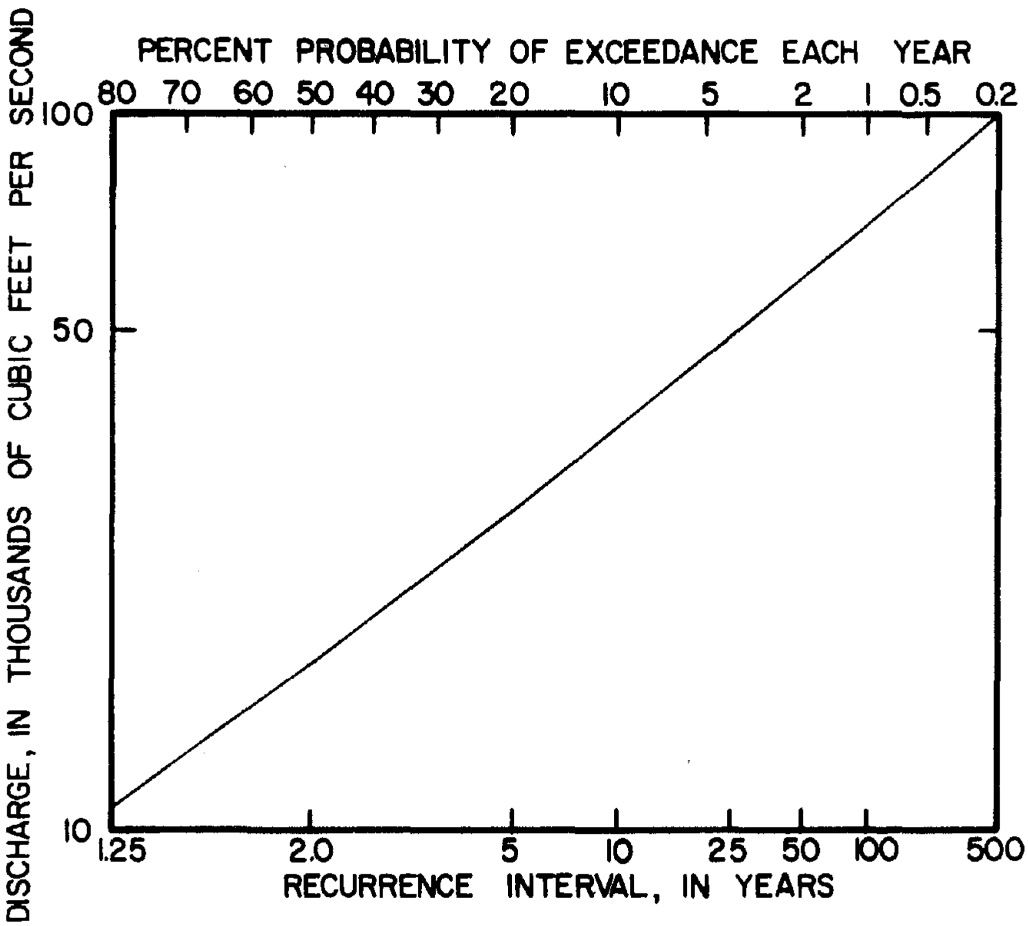

FIGURE B-7. MAGNITUDE AND FREQUENCY OF FLOODS, LEAF RIVER ABOVE BOWIE RIVER AT HATTIESBURG, MISS. (DRAINAGE AREA, 1100 SQUARE MILES.)

Hydrographs

The shape of the 1974 flood hydrograph was determined to be typical after comparison with other extreme floods and was used as a model for estimating hydrographs (figs. B-8 and B-9) for the flood peaks shown in table B-4. These hydrographs were used in calculating the time of overtopping (and resulting damages) of the various bridges and fills. They could also be used to calculate the depth of overtopping. However, in this case, the stage-discharge relation shown in figure B-10 was used to calculate the maximum depth of flow over the embankment. This method is based on the assumption that the depth of flow over the road is approximately equal to the difference between the water-surface elevations of the various discharges and the design discharge of the bridge and embankment combination. The water-surface elevations for the design discharge and for each of the various discharges are determined from the stage-discharge relation shown in figure B-10.

For example, the design discharge for the 440-ft bridge with an embankment elevation of 149 ft is 78,000 ft^3/s. As shown in figure B-11, the elevation from the stage-discharge relation is 148.3 ft. The elevation for the 2-percent-chance flood (110,000 ft^3/s) is 151.3 ft. The approximate overtopping depth is the difference in elevations or 151.3 - 148.3 = 3.0 ft. In a similar manner, the overtopping depth for the 1974 flood peak (121,000 ft^3/s) is 152.2 - 148.3 = 3.9 ft.

Natural Water-Surface Profile

A stage-discharge relation (fig. B-10) was developed using numerous current-meter discharge measurements obtained at this site during the period 1938-76. According to figure B-10, the elevation (at the downstream side of the main-channel bridge) corresponding to the 2-percent-chance flood (90,500 ft^3/s) is 149.8 ft, and the elevation corresponding to the 1-percent-chance flood (110,000 ft^3/s) is 151.5 ft. At bankfull stage (147 ft), the discharge is 66,000 ft^3/s, all of which is confined to the main channel. This discharge is a 5.6-percent-chance flood according to figure B-6.

The longitudinal water-surface profile (before highway construction) was computed by the step-backwater method using valley cross sections 2,600 and 6,000 ft downstream, at the centerline of the proposed crossing, and 5,300 and 11,000 ft upstream. Profiles for a flood barely overflowing the flood plain (68,000 ft^3/s), the flood of April 1974 (121,000 ft^3/s), and the 0.2-percent-chance flood (164,000 ft^3/s), are shown in figure B-12 along with the recovered flood profile of April 1974. The longitudinal profiles were computed using the U.S. Geological Survey step-backwater program (Shearman, 1976).

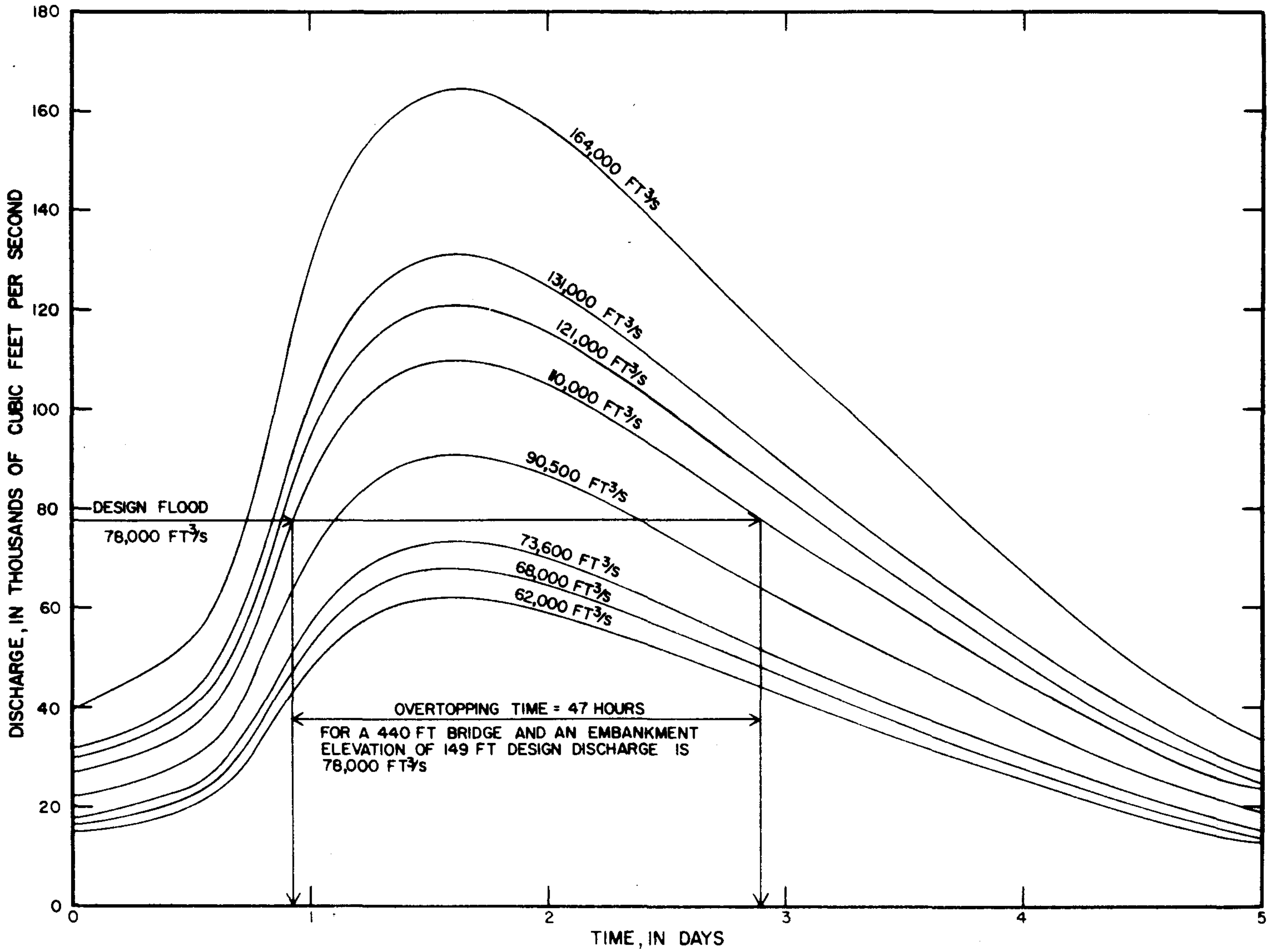

FIGURE B-8. DISCHARGE HYDROGRAPHS SHOWING THE CALCULATION OF THE OVERTOPPING TIME.

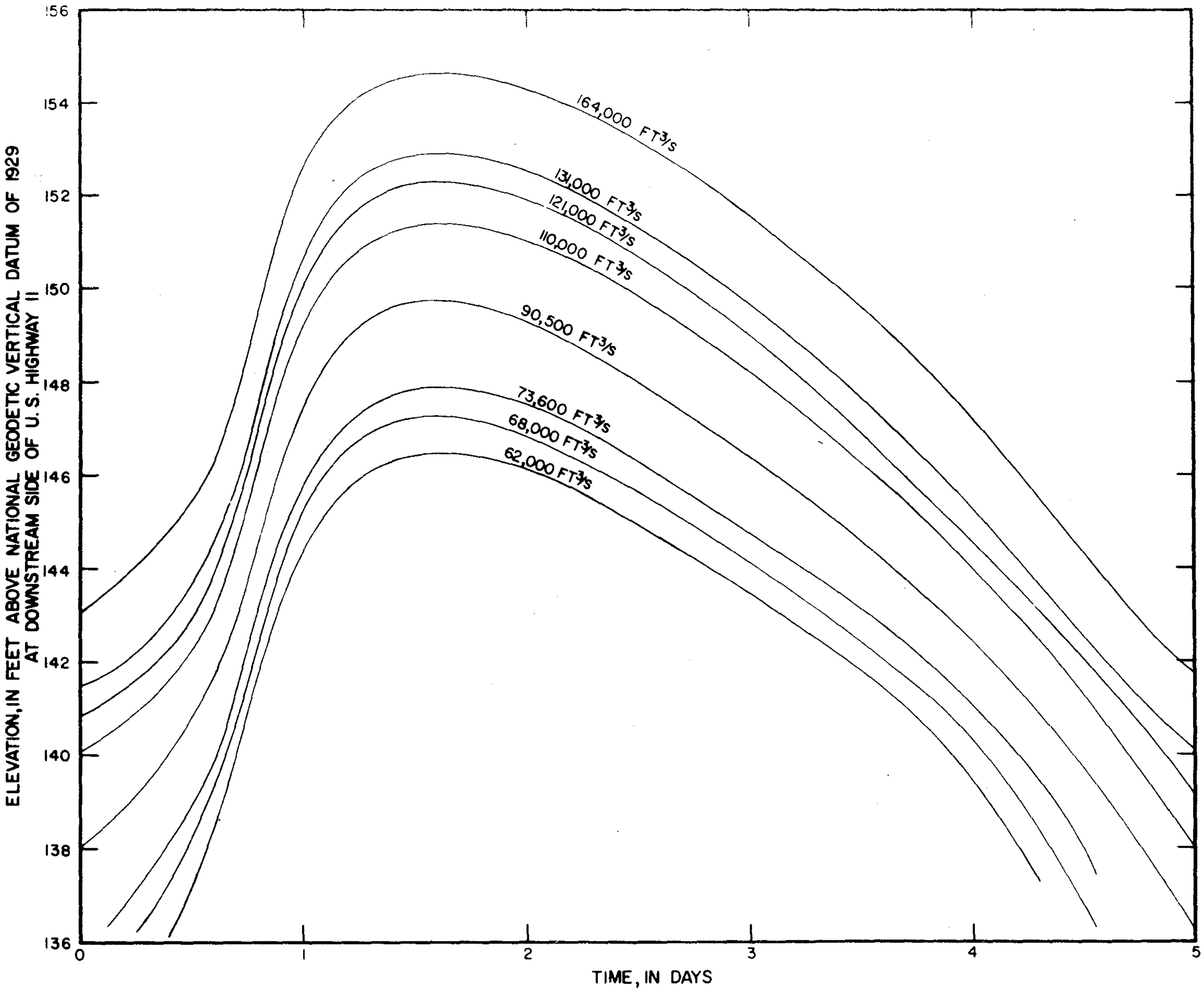

FIGURE B-9. STAGE HYDROGRAPH, LEAF RIVER AT U.S. HIGHWAY 11 AT HATTIESBURG, MISS.

Table B-4.--Flood peaks used in determining depths and periods of overtopping, Leaf River at U.S. Highway 11 at Hattiesburg, Mississippi

Flood discharge (ft^3/s)	Probability of occurrence (percent)
66,000 (bankfull)	5.6
68,000	5.0
72,000 (1961 flood)	4.2
73,600	4.0
90,500	2.0
110,000	1.0
121,000 (1974 flood)	0.63
131,000	0.50
164,000	0.20

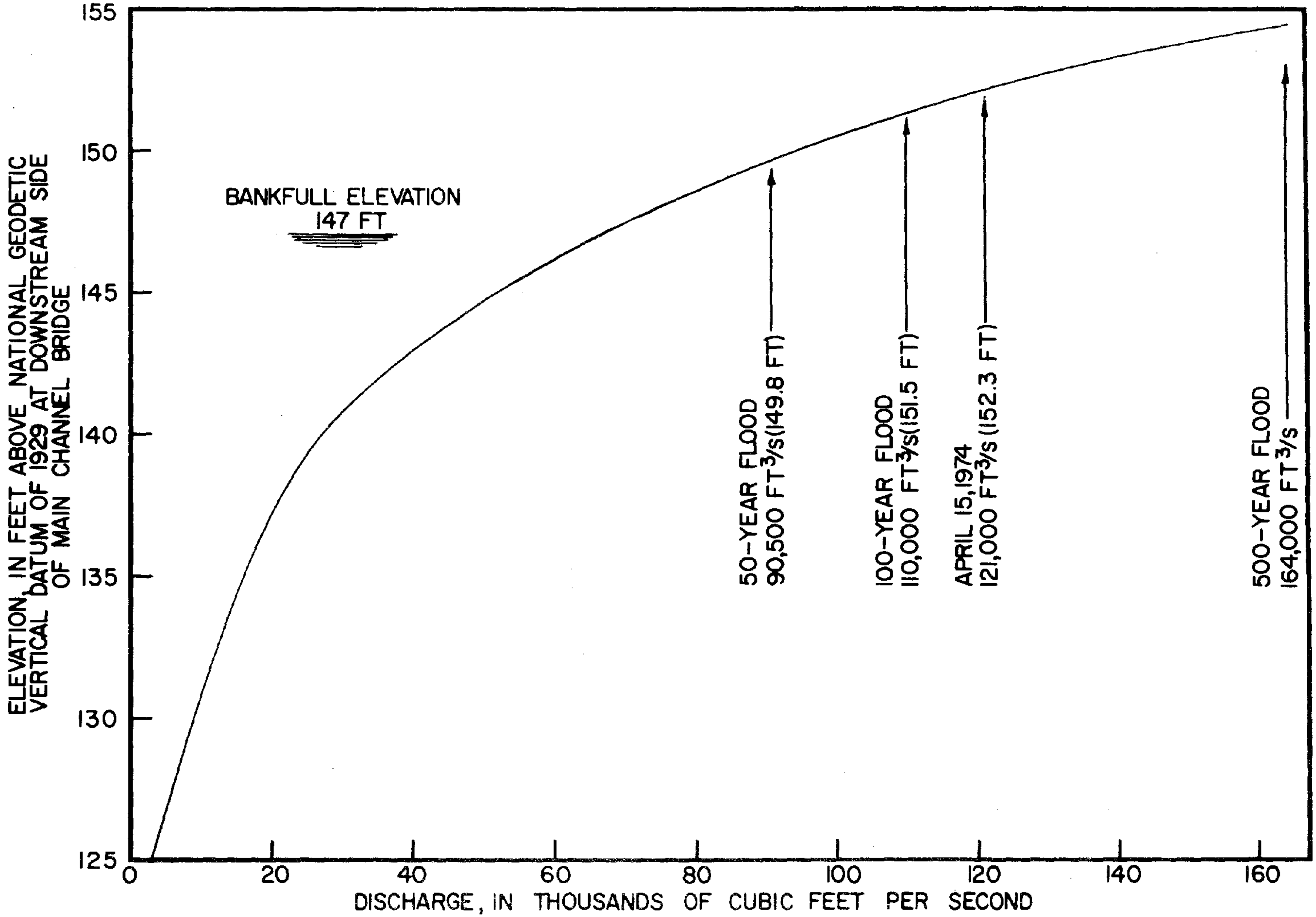

FIGURE B-10. STAGE-DISCHARGE RELATION, LEAF RIVER AT U.S. HIGHWAY 11 AT HATTIESBURG, MISS.

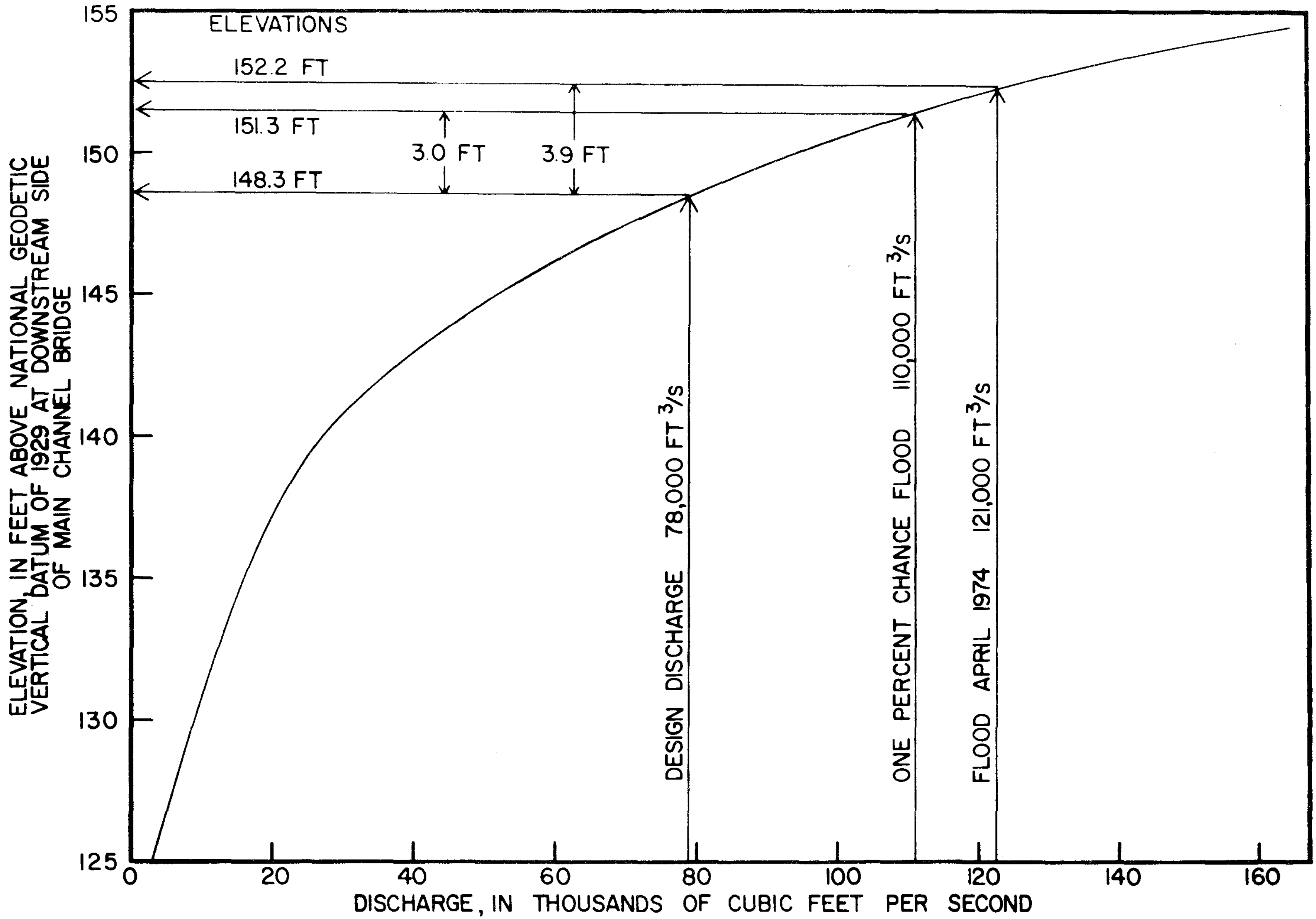

FIGURE B-11. USE OF STAGE-DISCHARGE RELATION TO CALCULATE THE APPROXIMATE OVERTOPPING DEPTH.

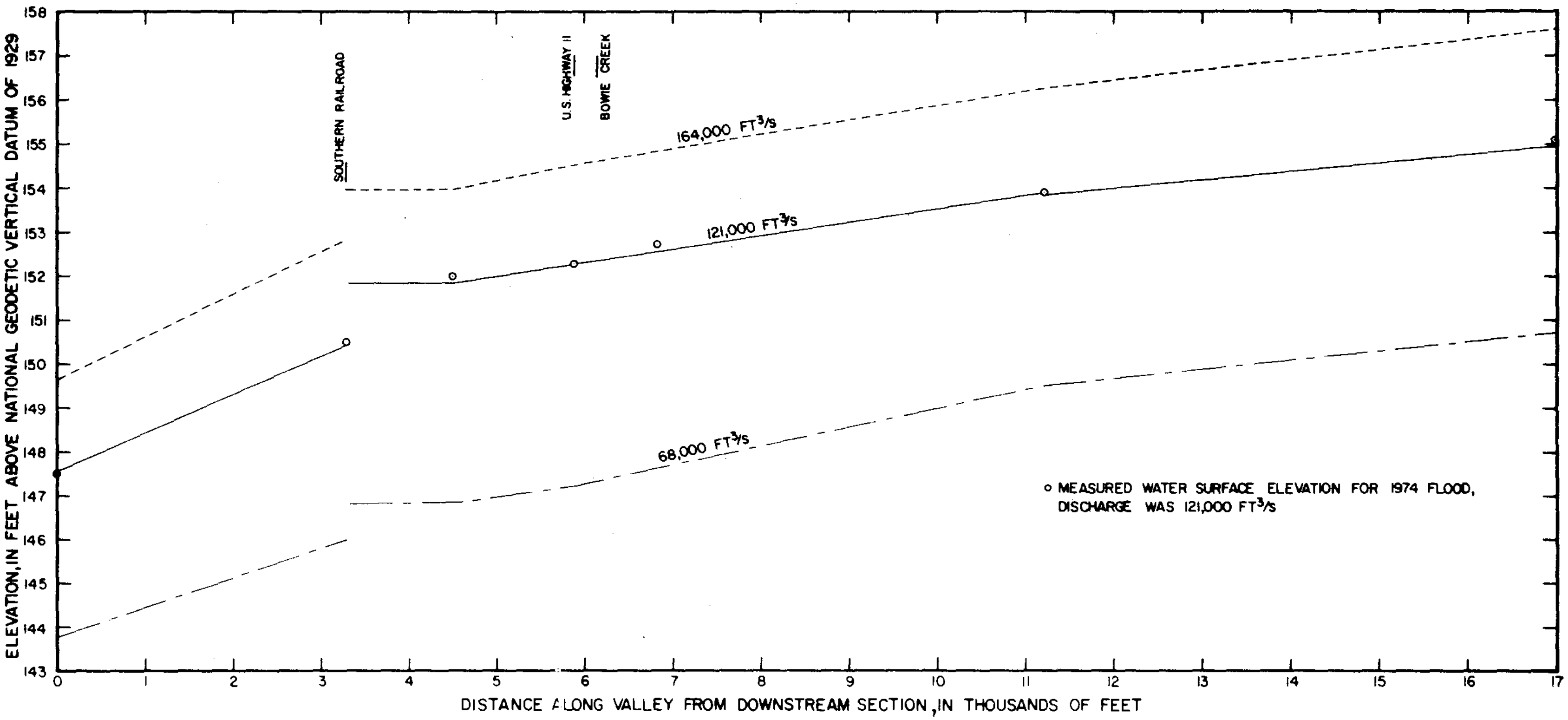

FIGURE B-12. TYPICAL COMPUTED NATURAL WATER SURFACE PROFILES.

Flow Distribution

The calculated flow distribution was based on the relative conveyance of the centerline cross section. The distribution is made by the step-backwater method computer program using Manning's equation. The results are shown in figure B-13.

During the 2-percent-chance flood, 54,700 ft^3/s approaches in the main channel, 22,500 ft^3/s is distributed on the west flood plain, and 13,300 ft^3/s is distributed on the east flood plain. During the 1-percent-chance flood, 58,200 ft^3/s approaches in the main channel, 32,000 ft^3/s is on the west flood plain, and 14,800 ft^3/s is on the east flood plain. According to these figures, approximately 53 percent of the 1-percent-chance flood is confined to the main channel.

Crest elevations of the April 1974 flood are shown on the valley cross section along the proposed highway alinement (fig. B-13). According to these elevations, the water surface at the west edge of the valley was 153.0 ft (0.7 ft above that at the main channel), and the water surface at the east edge of the valley was 151.4 ft (0.9 ft below that at the main channel).

The Southern Railroad bridge was not overtopped during the 1974 flood; therefore, the discharge which approached the alinement east of the railroad (sta. 49) flowed eastward through the Greens Creek railroad bridge (a mile north of the proposed highway alinement), flowed across the alinement and rejoined the proposed highway alinement, flowed across the alinement and rejoined Leaf River half a mile downstream. Similar flow would occur during the 2-percent- and 1-percent-chance floods.

Special Considerations

Drift

Drift at this site should be minor according to observations of the U.S. Geological Survey near the crests of the 1961 and 1974 floods during which time little or no drift was observed.

Soils

The channel and the banks of Leaf River have been stable in the vicinity of U.S. Highway 11 since gaging records began in 1938. Sand and gravel operations in the valleys of both Leaf and Bowie Rivers just upstream from the crossing have existed for many years but with little adverse effect in the vicinity of U.S. Highway 11 where the channel is composed primarily of dense sand and gravel with silty clay lining the flood plain.

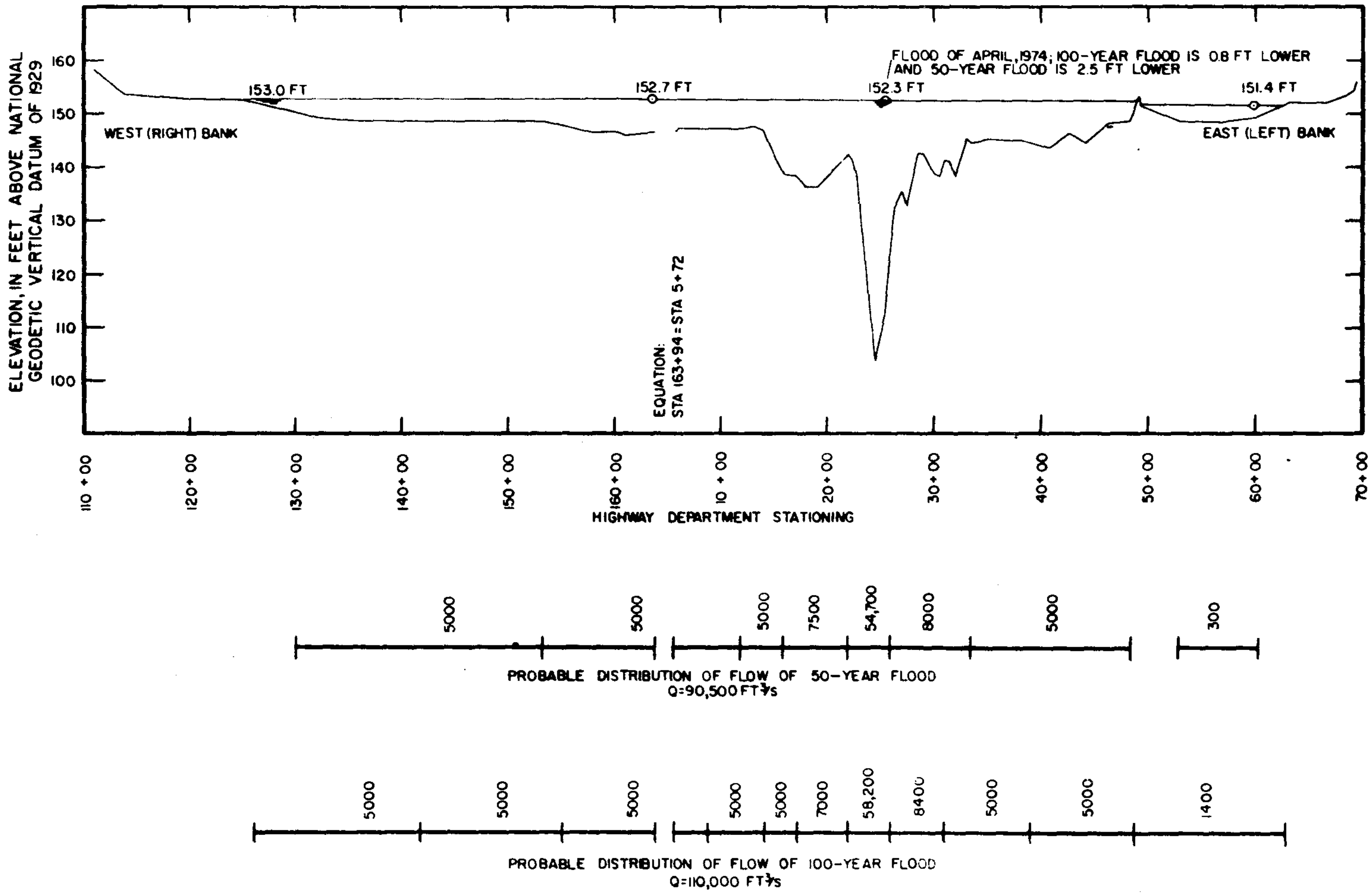

FIGURE B-13. PROBABLE DISTRIBUTION OF FLOW, LEAF RIVER AT U.S. HIGHWAY 11 AT HATTIESBURG, MISS.

Turbulence created by the mixing of the water from Leaf River and Bowie River creates a potential scour problem just upstream from U.S. Highway 11. The surveyed centerline of the proposed crossing about 150 ft upstream from the existing crossing shows a bottom elevation of about 104 ft compared to about 116 ft at present U.S. Highway 11. These data indicate that the proposed crossing may be in the edge of a scour hole created by the confluence of the two streams.

Spur Dikes

Spur dikes are needed at this site for embankment elevations of 148 ft and above because of the high percentage of flow on the flood plains for these floods. Analyses of embankments using methods described by Bradley (1973) indicated that 150-ft dikes are needed at both abutments for all bridges with embankment elevations of 151 ft or greater. Dikes either 100- or 150-ft long are needed for bridges less than 1,600 ft with embankment elevations of 148 ft. The costs of these dikes were estimated by the Highway Department and are incorporated in the construction costs.

Channel Protection

Channel protection for both abutments is necessary for bridges with average velocities greater than 6 ft/s because the sand and gravel banks of the channel are subject to scour. Extensive protection works at both abutments ($500,000) were included for some bridges shown in table B-2.

Alternative Bridges Considered in Risk Analysis

Six bridges were analyzed in the risk analysis. The low chord in each case was set so that it was not submerged for the hydraulic conditions considered in this report. The six combinations include:

1. 280-ft bridge (three spans: 50 ft, 180 ft, 50 ft)
2. 440-ft bridge (three spans: 130 ft, 180 ft, 130 ft)
3. 600-ft bridge (five spans: 130 ft, 180 ft, 130 ft, 2 at 80 ft)
4. 800-ft bridge (nine spans: 130 ft, 180 ft, 130 ft, 2 at 100 ft, 4 at 40 ft)
5. 1,200-ft bridge (19 spans: 130 ft, 180 ft, 130 ft, 2 at 100 ft, 14 at 40 ft)
6. 1,600-ft bridge (29 spans: 130 ft, 180 ft, 130 ft, 2 at 100 ft, 24 at 40 ft)

Five embankment elevations were considered for each bridge. The elevations were varied from 147 to 155 ft in 2-ft increments. Therefore, 30 possible combinations of bridges and embankments were considered at this site.

Water-Surface Profiles

High-water elevations were obtained after the 1974 flood at numerous points in the vicinity of the proposed crossing and throughout the reaches extending 6,000 ft downstream and 11,000 ft upstream. These elevations were used to develop the natural 1974 flood profile throughout this 17,000-ft distance. The present highway is constructed at flood-plain level and creates no appreciable backwater. The 1974 flood profile was modeled with adjustment of roughness coefficients, using the step-backwater program (Shearman, 1976) for the existing U.S. 11 crossing and the 1974 peak discharge. Comparisons of the measured and computed 1974 flood profiles are shown in figure B-14.

With this satisfactory agreement for the 1974 flood, the hydraulics were computed using each of the 30 bridge and embankment combinations described previously. Typical computed profiles are shown in figure B-15. A summary of the water-surface elevations used to compute damage caused by backwater for a bridge length of 440 ft and embankment elevation of 149 ft is presented in table B-5.

RISK ANALYSIS

A flood passing through this bridge may cause several types of damage. These include traffic delay caused by flow over the embankment and embankment washout, damage caused by increased backwater, and the cost of embankment repairs. The bridge is assumed not to be damaged during any event since the piers and foundations are set at sufficient depth so that they will not be undermined by scour.

After the flood profiles for the natural and constricted conditions were computed, the resulting stages, times, and depths were used in the damage relations defined previously to compute damages.

The total construction costs vary with bridge length and embankment elevation as shown in figure B-16. The curves are not smooth because large costs of channel protection are eliminated from the longer bridges. The 280-ft bridge costs slightly more than the 440-ft bridge for each embankment elevation.

Damage due to each flood peak was computed for each combination and entered in table B-6 as shown. The risk or expected damage was computed by averaging the damage over two adjacent discharges, multiplying by the

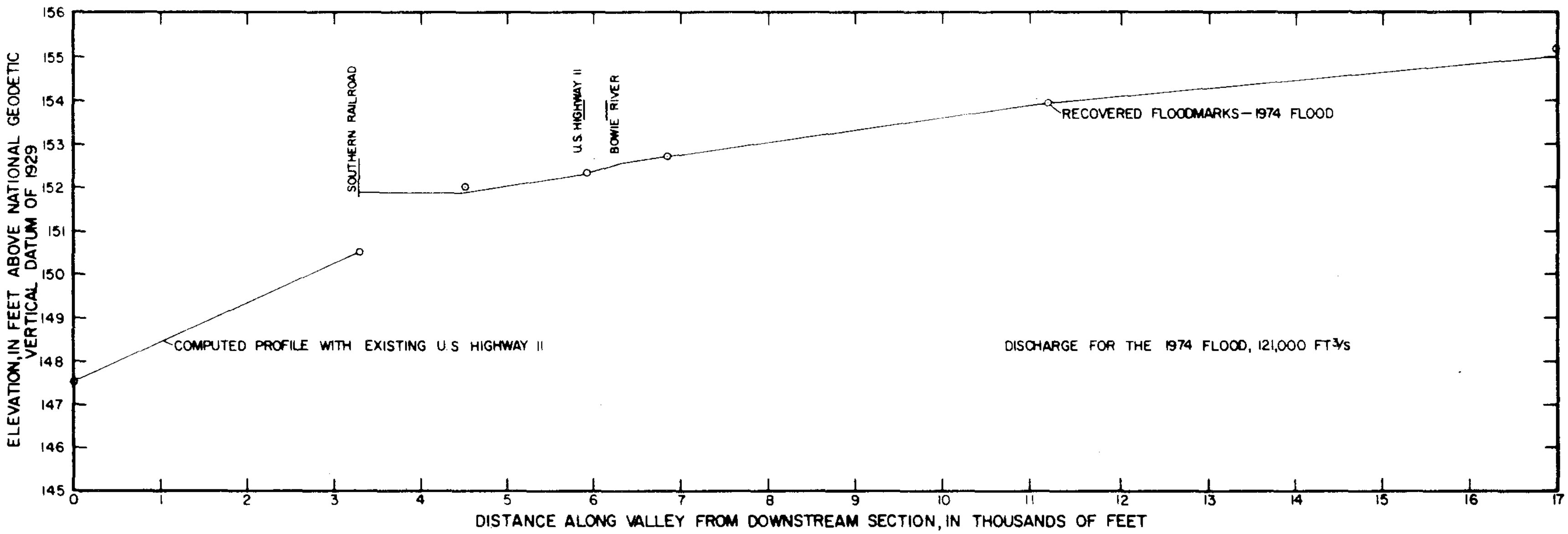

FIGURE B-14. COMPARISON BETWEEN THE COMPUTED AND RECOVERED PROFILES FOR THE 1974 FLOOD.

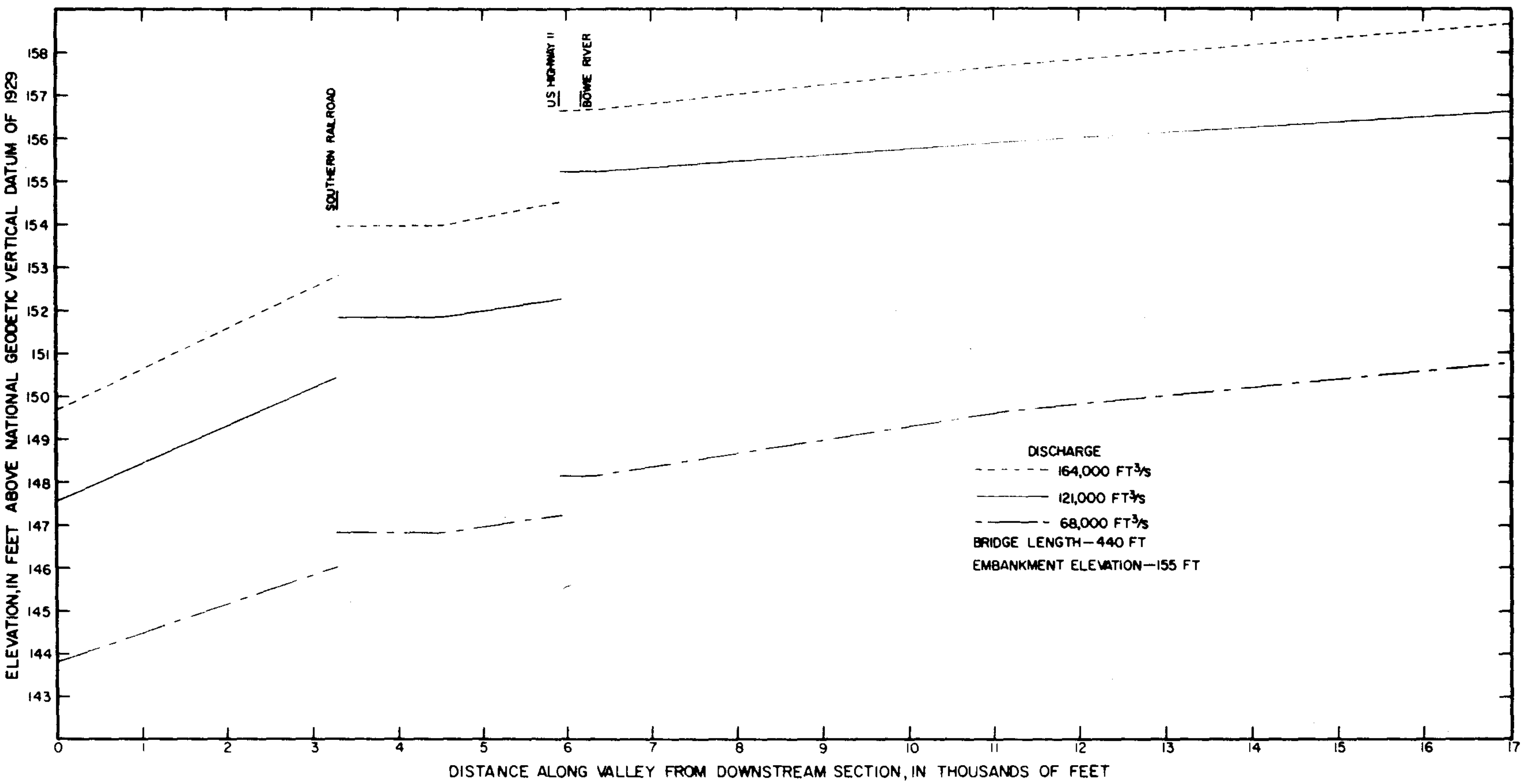

FIGURE B-15. TYPICAL COMPUTED WATER SURFACE PROFILES WITH THE BRIDGE.

Table B-5.--Summary of water-surface elevations used to compute damage caused by backwater for a bridge length of 440 ft and an embankment elevation of 149 ft

Flood	Water-surface elevations, ft			
	Station 6340		Station 11200	
	With bridge	Natural	With bridge	Natural
51,000	145.58	145.58	147.23	147.23
54,800	146.18	146.18	147.76	147.76
62,000	147.34	147.34	148.96	148.83
68,000	148.17	147.99	149.70	149.45
73,600	148.99	148.56	150.40	149.99
90,500	150.21	150.21	151.59	151.53
110,000	151.84	151.84	153.09	153.09
121,000	152.68	152.68	153.82	153.85
131,000	153.39	153.39	154.59	154.59
164,000	154.40	154.40	155.85	155.85

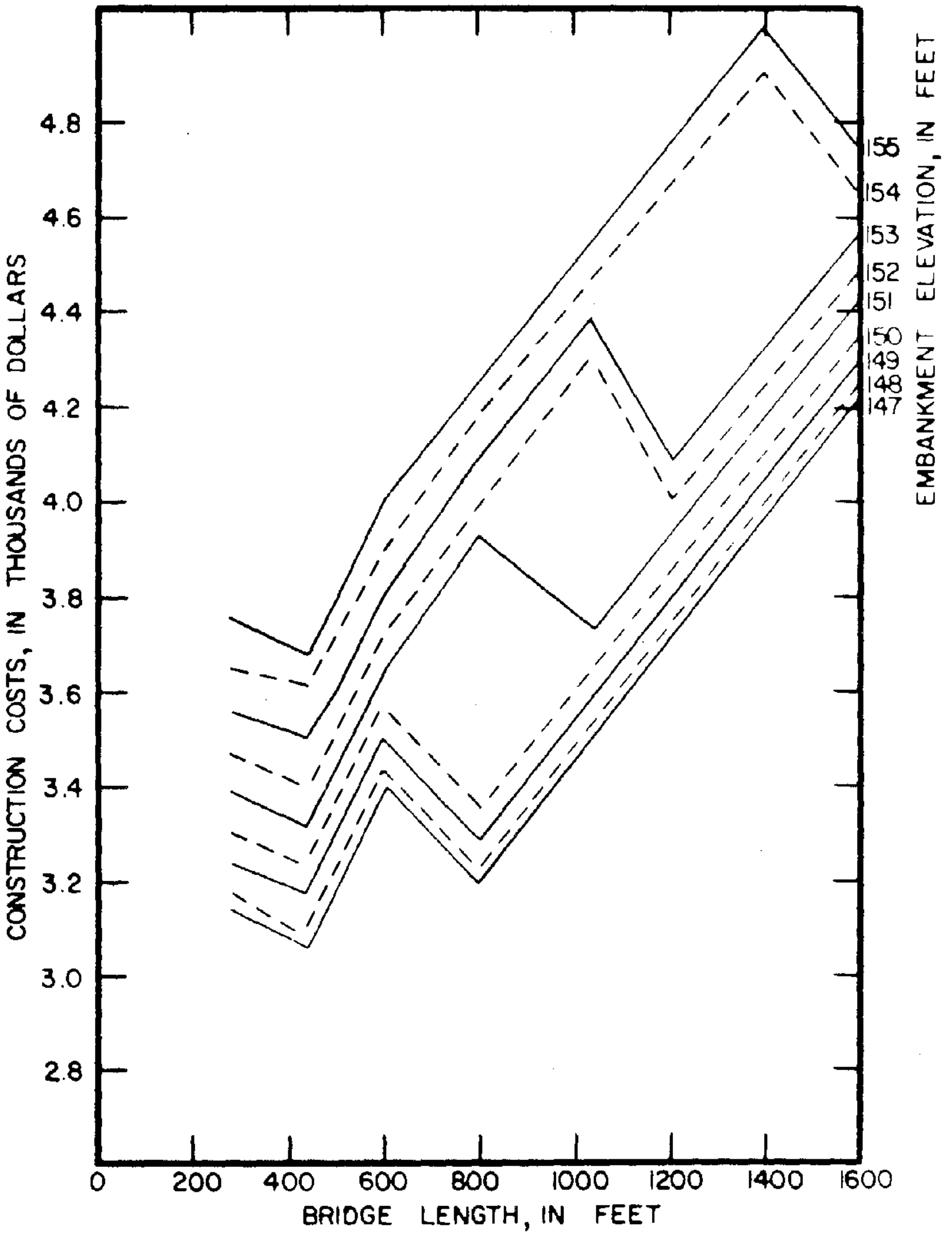

FIGURE B-16. RELATION BETWEEN CONSTRUCTION COSTS AND BRIDGE LENGTH FOR PROPOSED URBAN HIGHWAY.

Table B-6.--Summary of risk analysis

Site: Leaf River at Hattiesburg, Miss.

Design Discharge: 78,000 ft^3/s

Embankment Height: 149 feet

Bridge Deck Length: 440 feet

Estimated Capital Cost: $3,168,000

Estimated Service Life: 25 years

Interest Rate: 5 percent

Annual Capital Cost: $224,770

Flood ft^3/s	Flood probability	Maximum depth over roadway, feet	Duration of flow over the road, hours	Time road is closed for repairs, hours	Cost in dollars: Traffic interruption	Cost in dollars: Backwater damage	Cost in dollars: Overtopping damage to embankment	Cost in dollars: Other damage	Total damage, dollars	Average damage for interval, dollars	Probability interval, ΔP	Risk, dollars
51,000	0.111											
54,200	.100					Computed					0.011	
62,000	.067					by					.033	
68,000	.050					offline					.017	
73,600	.040					program					.010	
78,000	.034	0	0		0	which	0		0		.006	
90,500	.020	1.3	31	0	100,891	output	32,940		133,831	66,915	.014	937
110,000	.010	3.0	47	0	152,963	risk	323,910		476,873	305,352	.010	3,054
121,000	.00625	3.9	54	72	410,072	due to	625,860		1,035,932	756,402	.00375	2,836
131,000	.0050	4.5	49	72	426,345	back-	914,524		1,340,869	1,188,400	.00125	1,485
164,000	.002	6.3	73	72	471,909	water	914,524		1,386,433	1,363,651	.003	4,091
Ultimate									1,386,433	1,386,433	.002	2,773
				Risk Subtotals	$5,845	$3,600	$9,331					

Total $15,176

Backwater Risk 3,600

Total Risk $18,776

Annual Capital Cost 224,770

Total Expected Project Cost $243,546

increment of probability, and summing over all possible combinations. The backwater damages were computed as a total for all floods. The results of the risk analysis and damage computations are summarized in figures B-17 - B-19A. The annual construction costs were computed assuming a 5-percent interest rate and a 25-year project life. The resulting capital recovery factor is 0.070952. (Multiply the total construction cost by the capital recovery factor to get the annual cost.) The results of all the risk and capital cost computations are summarized in table B-7.

The annual construction costs and risk vary with embankment elevation and bridge length as shown in figure B-17. The risk is greatest for the 280-ft bridge and the 155-ft embankment elevation. The severe contraction of the channel causes significant backwater damage and the high velocities in the bridge opening will result in the loss of the channel protection. Similar results are shown in figure B-18 where annual construction costs and risk are related to the bridge length at various embankment elevations.

The total expected project cost (the sum of risk and annual construction costs) are shown in figure B-19A as a function of embankment elevation and bridge length. The 440-ft-long bridge is the least costly with embankment elevations of 147 and 149 ft.

The design discharge for each of the combinations considered is shown in figure B-19B. For the 440-ft bridge, the design discharges are 60,000 ft^3/s and 78,000 ft^3/s for embankment elevations of 147 ft and 149 ft.

Since the total expected project cost for the 440-ft-long bridge changes relatively little between embankment elevations of 147 ft and 149 ft, the design discharge of the least costly structure probably lies between 68,000 ft^3/s and 78,000 ft^3/s with an exceedance probability between 7.7 and 3.3 percent.

Figure B-19 along with the discharge-frequency curve in figure B-6 is used to develop figure B-20. In figure B-20, total expected project cost is related to the exceedance probability of the design discharge and bridge length. For a constant exceedance probability, the total expected project cost may vary with bridge length but not necessarily in direct proportion to bridge length. In addition, for exceedance probabilities smaller than 2 percent, the total expected project costs for the 440-ft-, 600-ft-, and 800-ft-long bridges are nearly the same.

The conclusion that the 440-ft-long bridge is the least costly structure is based on existing conditions. In order to determine the effect of changing property values and traffic on the conclusion, figures B-21 and B-22 were prepared. In figure B-21, the total expected

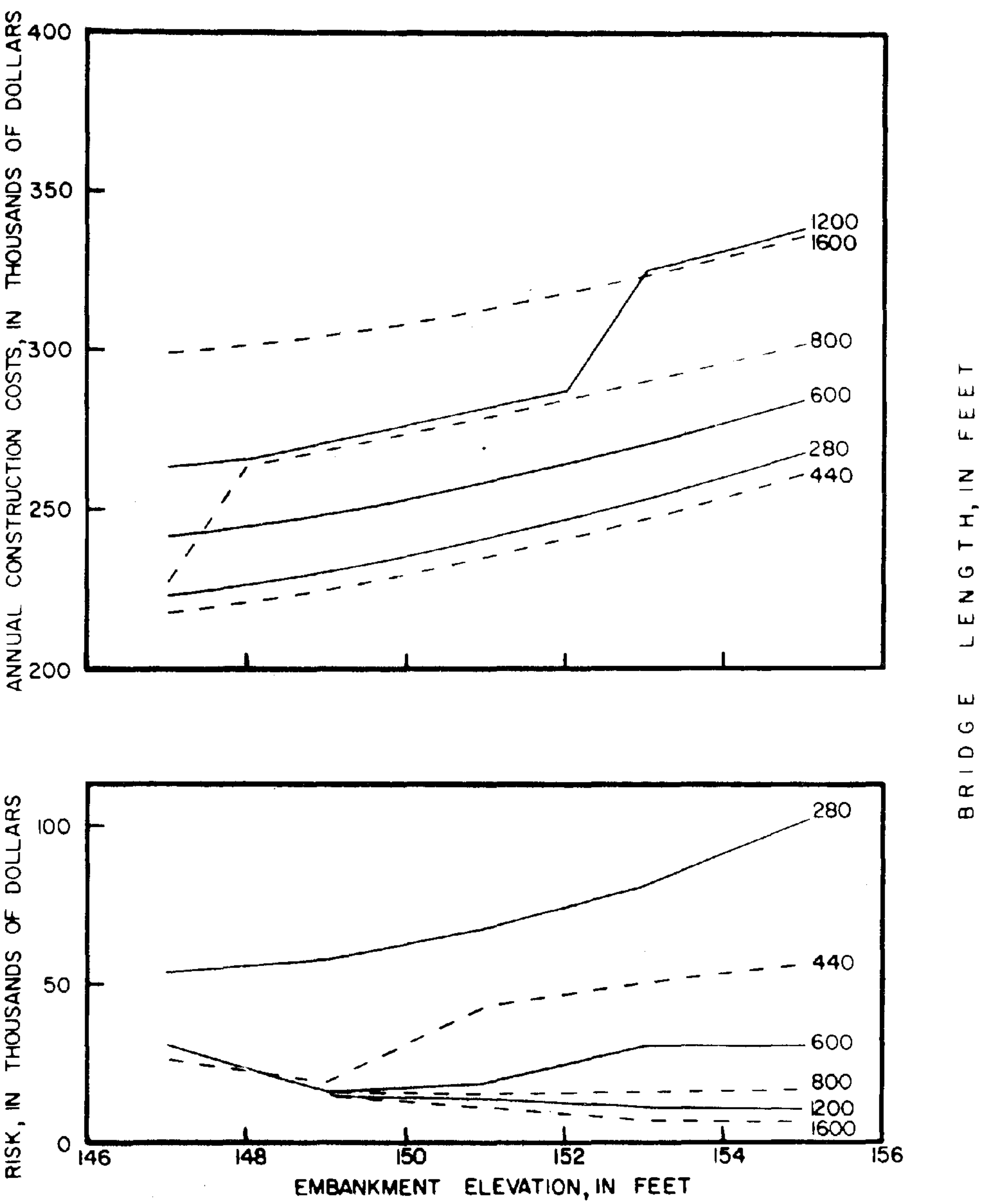

FIGURE B-17. ANNUAL CONSTRUCTION COST AND RISK AS A FUNCTION OF EMBANKMENT ELEVATION AND BRIDGE LENGTH.

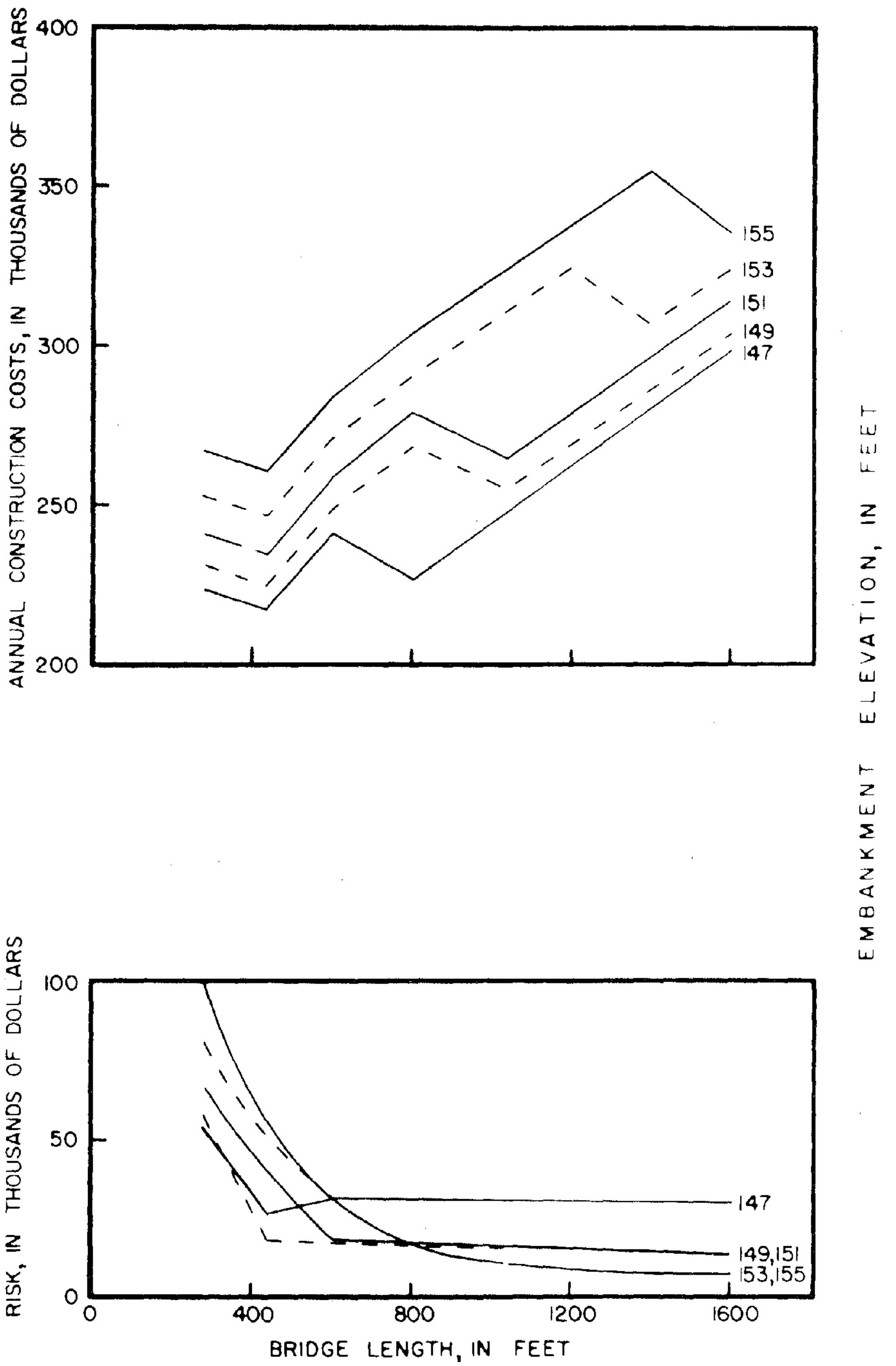

FIGURE B-18. ANNUAL CONSTRUCTION COST AND RISK AS A FUNCTION OF EMBANKMENT ELEVATION AND BRIDGE LENGTH.

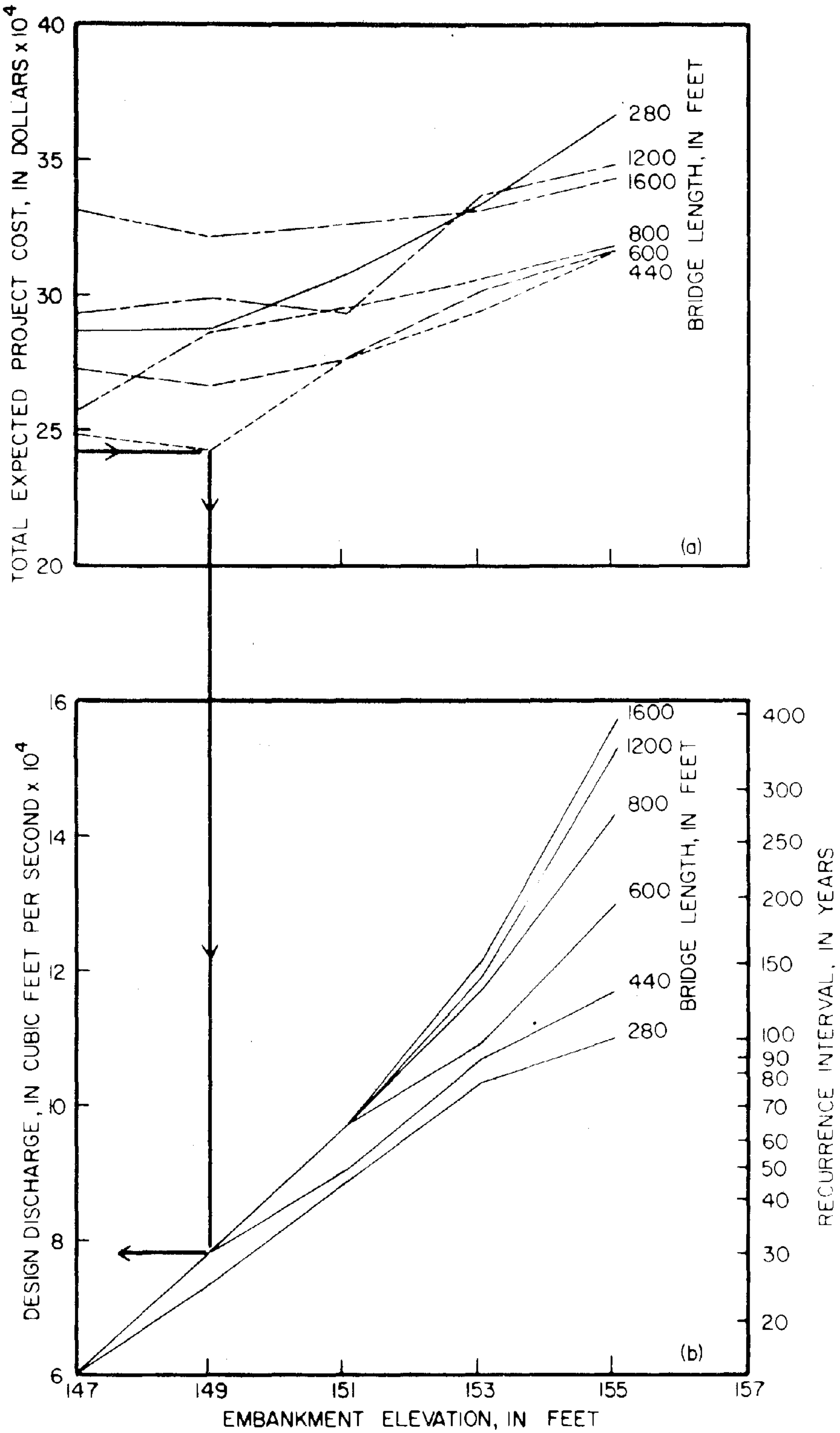

FIGURE B-19. SELECTION OF THE DESIGN DISCHARGE, BRIDGE LENGTH, AND EMBANKMENT ELEVATION GIVEN THE TOTAL EXPECTED PROJECT COSTS.

Table B-7.--Summary of risk and capital cost data

Bridge length (ft)	Embankment elevation (ft)	Design discharge (ft^3/s)	Exceedance probability of the design discharge	Annual risks					Annual capital cost	Total expected project cost
				Traffic interruption	Backwater damage	Embankment damage	Other[1]	Total risk		
280	147	60,000	.075	16,823	0	14,017	33,500	64,340	222,647	286,987
	149	73,600	.040	6,647	6,000	11,884	33,500	58,031	230,097	288,128
	151	88,500	.023	3,374	22,500	9,030	33,500	68,404	240,669	309,073
	153	103,600	.012	1,352	40,500	6,178	33,500	81,530	252,802	334,332
	155	110,000	.010	1,032	60,000	5,749	33,500	100,281	266,780	367,061
440	147	60,000	.075	16,823	0	14,017	0	30,840	217,397	248,237
	149	78,000	.034	5,845	3,000	9,331	0	18,776	224,776	243,552
	151	90,100	.020	2,276	14,700	8,042	17,000	42,018	235,206	277,224
	153	106,900	.011	1,189	27,400	2,279	17,000	47,868	247,268	295,136
	155	116,900	.008	823	34,800	3,438	17,000	56,061	261,103	317,164
600	147	60,000	.075	16,823	0	14,017	0	30,840	241,592	272,432
	149	78,000	.034	5,833	2,300	9,327	0	17,460	248,829	266,289
	151	97,000	.016	2,576	9,500	6,546	0	18,622	258,975	277,597
	153	109,000	.011	1,109	18,000	4,364	7,500	30,973	270,824	301,797
	155	130,000	.005	488	21,900	814	7,500	30,702	284,376	315,078
800	147	60,000	.075	16,823	0	14,017	0	30,840	226,763	257,603
	149	78,000	.034	5,833	2,000	9,327	0	17,160	269,192	286,352
	151	97,000	.016	2,576	7,000	6,546	0	16,122	279,125	295,247
	153	117,000	.008	816	12,400	2,513	0	15,729	290,619	306,348
	155	143,500	.004	259	14,600	456	0	15,315	303,178	318,498

Table B-7.--Summary of risk and capital cost data--continued

Bridge length (ft)	Embankment elevation (ft)	Design discharge (ft^3/s)	Exceedance probability of the design discharge	Annual risks					Annual capital cost	Total expected project cost
				Traffic interruption	Backwater damage	Embankment damage	Other[1]	Total risk		
1,200	147	60,000	.075	16,823	0	14,017	0	30,840	263,090	293,930
	149	78,000	.034	5,833	1,500	9,327	0	16,660	282,318	298,978
	151	97,000	.016	2,576	6,000	6,546	0	15,122	278,912	294,034
	153	119,000	.007	729	9,100	2,015	0	11,844	325,315	337,159
	155	153,200	.003	160	10,400	63	0	10,623	337,802	348,425
1,600	147	60,000	.075	16,823	0	14,017	0	30,840	299,417	330,257
	149	78,000	.034	5,833	1,000	9,327	0	16,160	305,306	321,466
	151	97,000	.016	2,576	4,000	6,546	0	13,122	314,246	327,368
	153	121,000	.006	660	6,200	699	0	7,559	324,463	332,022
	155	157,100	.002	121	6,800	19	0	6,940	336,242	343,182

[1]The cost in this column is the assumed loss of channel protection when velocities in the bridge opening exceed 9 ft/s.

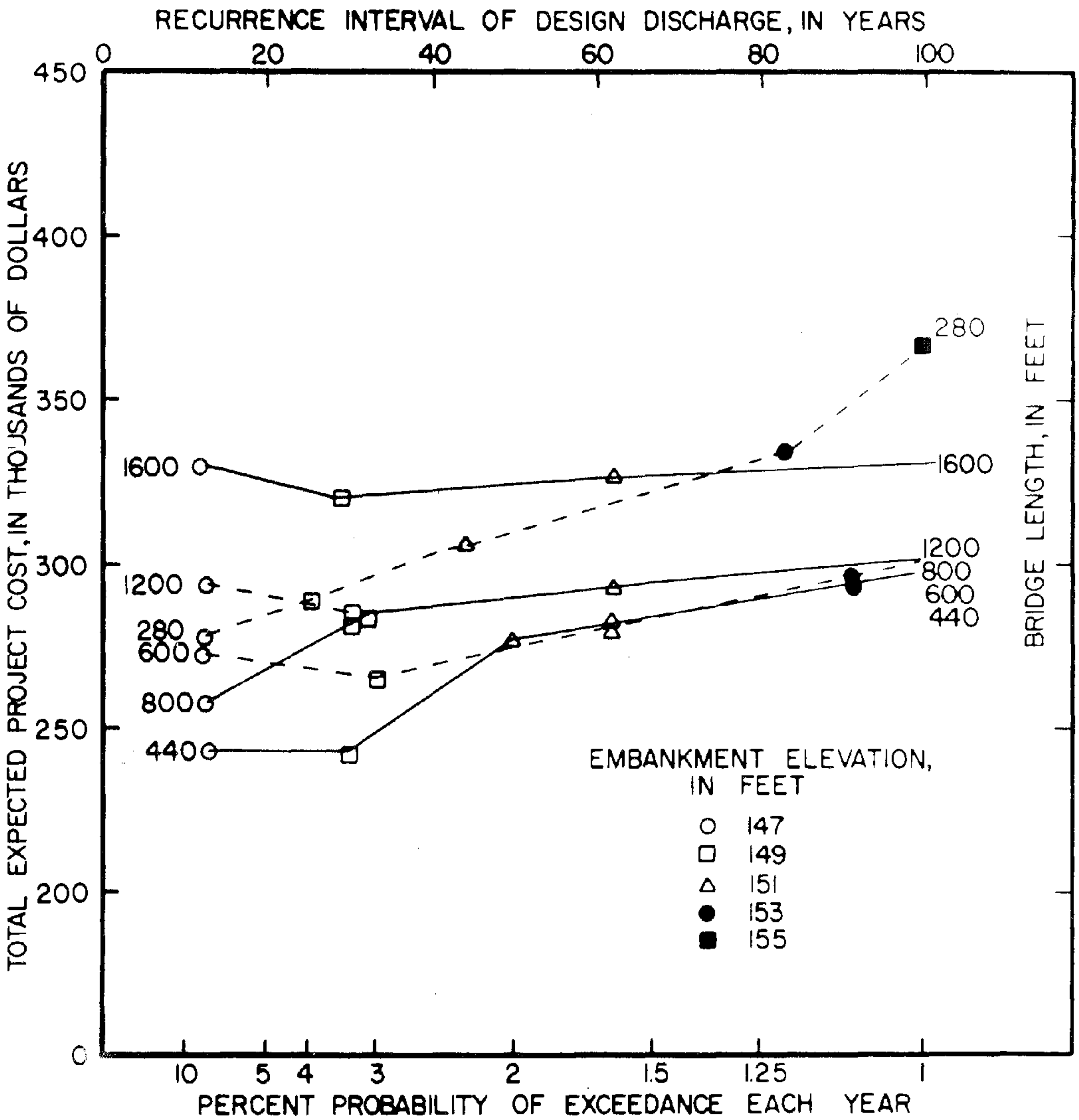

FIGURE B-20. TOTAL EXPECTED PROJECT COST AS A FUNCTION OF RECURRENCE INTERVAL OF DESIGN DISCHARGE AND BRIDGE LENGTH.

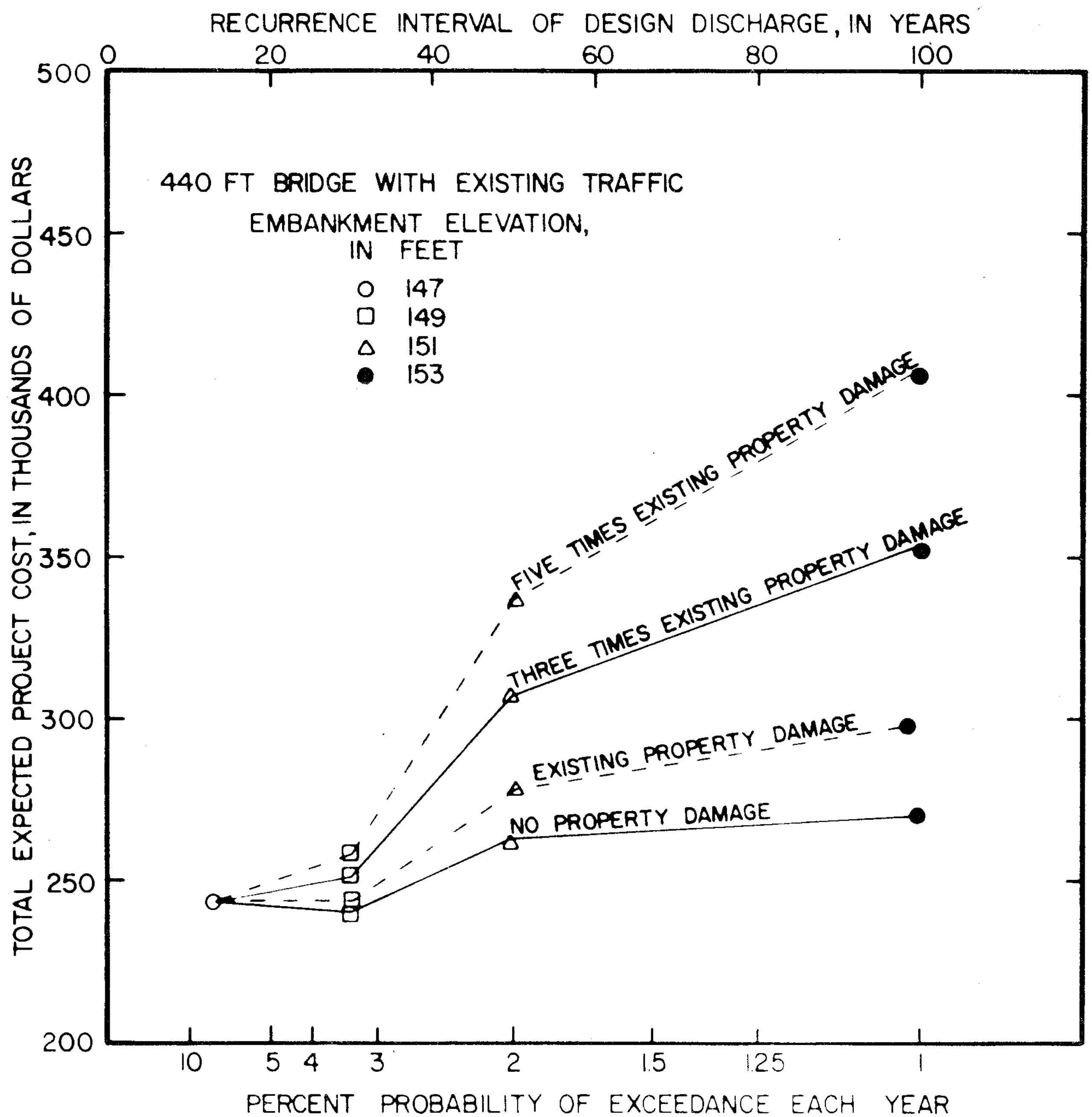

FIGURE B-21. EFFECT OF PROPERTY DAMAGE VARIATION ON THE RECURRENCE INTERVAL OF THE DESIGN DISCHARGE.

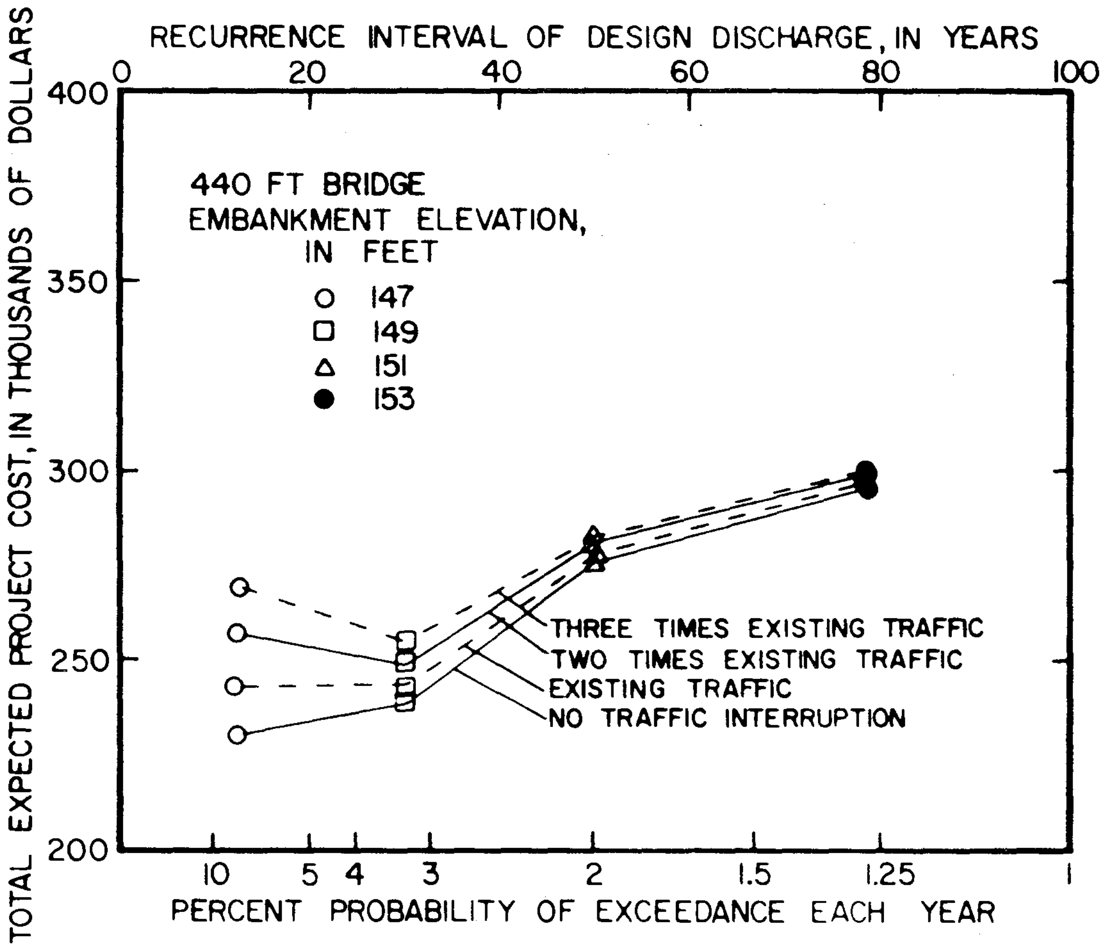

FIGURE B-22. EFFECT OF INCREASED TRAFFIC ON THE RECURRENCE INTERVAL OF THE DESIGN DISCHARGE.

project cost is related to flood exceedance probability for four values of property--including negligible property damage (a rural-like site), the existing property damage, three times the existing property damage, and five times the existing property damage. As the value of the property that is damaged increases, the design flood exceedance probability decreases. The risk is relieved by lowering the embankment elevation to the flood-plain elevation. In effect, when there is no embankment, there is no backwater damage caused by the bridges.

A similar comparison is made in figure B-22 for traffic. Total expected project cost is related to the flood exceedance probability for no traffic interruption, traffic interruption based on ADTE, two times ADTE interruption, and three times ADTE interruption. The risk due to traffic interruption is relieved by raising the embankment elevation, which increases the exceedance probability of the design discharge.

As property values and traffic volumes increase, the penalty for selecting too low a design flood exceedance probability increases. For example, designing for the 2-percent-chance flood instead of the 7.7-percent-chance flood with three times the existing property damage increases the total expected project cost $55,000 per year or a 23-percent increase from the least costly alternative. When dramatic changes in land use that increase potential damage or change in traffic volumes are anticipated and when the solution seems very sensitive to these changes, the risk analysis should be recomputed based on the future property values or traffic volumes.

SUMMARY AND CONCLUSIONS

The design procedure outlined in this report which uses a risk analysis indicates that a 440-ft-long bridge with a minimum embankment elevation of 147 ft to 149 ft is the minimum cost bridge for the factors considered. The factors considered include construction cost, backwater damage, traffic interruption, and embankment repairs. The bridge includes a 100-ft-long spur dike on each side of the channel and channel work to protect the abutments. An embankment elevation of 147 ft will be overtopped by the 7.7-percent-chance flood whereas the embankment elevation of 149 ft will be overtopped by the 3.3-percent-chance flood. A comparison of the risk components for these two bridges is made in table B-8. Even though the total expected project cost is the same for these two bridges, there are differences in traffic interruption, backwater damage, and expected embankment damage owing to overtopping.

The final choice of a design flood is beyond the scope of this example report. The final decision will include a consideration of the information developed by a report such as this, along with budgetary constraints, the need for emergency supply and evacuation routes, the need for emergency vehicle access, and other site specific considerations.

Table B-8.--Comparison of the risk components for a 440-ft-long bridge with a 147 ft and 149 ft embankment elevation

Risk component	Embankment elevation (ft)	
	147	149
Traffic interruption	$13,095	$5,845
Backwater damage	0	3,600
Embankment repairs	13,589	9,331
Total risk	$26,684	$18,776

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U.S. GOVERNMENT PRINTING OFFICE : 1980 O-622-537.1204